HYDRAULIC MODEL STUDY OF THE MORNING GLORY SPILLWAY FOR DILLON DAM

Prepared for
TIPTON AND KALMBACH, INC.,
Denver, Colorado

by

SUSUMU S. KARAKI

COLORADO STATE UNIVERSITY RESEARCH FOUNDATION
Civil Engineering Section
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FOREWORD

The results of a hydraulic model study of the Dillon Dam Spillway are presented in this report. The study was conducted in the Hydraulics Laboratory of Colorado State University, which is under the general technical and administrative supervision of Dr. A. R. Chamberlain, Chief of the Civil Engineering Section. Technical advice was received in the early stage of the study from A. J. Peterka, of the United States Bureau of Reclamation who was consulting engineer to Colorado State University on model studies.

Dillon Dam and the appurtenant works is being designed by the consulting engineering firm of Tipton and Kalmbach, Inc. It was for this firm that this model study was conducted.

Pertinent information for the model study and this report was derived from the reports -- "Dillon Dam and Reservoir - Volume I - Text and Volume II - Appendices", December, 1957, and the "Supplemental Report on Dillon Dam and Reservoir", November, 1958, written by Tipton and Kalmbach, Inc. The first two volumes contain the basic hydrologic data of the watershed and geologic information of the dam site as well as a summary of earlier investigations and feasibility studies conducted by other engineering firms. The third volume contains the basic information used to develop the preliminary design of the dam. The contents of the latter text is based on the premise that the dam will be constructed in two stages, as the need for water arises for the City of Denver. It is the same premise upon which the writer has based the model study and this report.

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SYNOPSIS

The purpose of this model study was to check various features of the preliminary design of the morning glory spillway and tunnel and to make experimental modifications as necessary to produce a design that would perform adequately under the expected conditions of flow. The studies were made in a 1:31.31 scale model.

During the period of testing, the location of the morning glory spillway was changed twice. For each change the principal objectives of the study were:

1. To determine an economical excavation for the spillway approach.
2. To check the performance of the preliminary spillway crest.
3. To develop the junction of the spillway tunnel with the outlet works tunnel.
4. To design, through experimentation, a satisfactory energy dissipator at the outlet of the tunnel.

Each of these objectives required extensive study before a satisfactory solution was found.

Excessive negative pressures were measured in the throat and vertical shaft of the morning glory spillway of the preliminary design which was corrected by use of a deflector above the vertical bend.

Concentration and non-uniform distribution of flow on the crest was solved by installation of four piers on the crest. Excessive turbulence and erosive forces were noted at the horizontal junction of the spillway and outlet tunnels of the preliminary design. Development of a junction below the vertical bend of the spillway shaft solved the problem.

At the tunnel outlet, the model study indicated that a deflector bucket would not function satisfactorily for discharges less than 1000 c.f.s. Damage would have resulted from the scour adjacent to the end of the structure. The final design of the energy dissipator which resulted from the model study, was a combination hydraulic jump stilling basin and deflector bucket. Discharges up to 3000 c.f.s. were contained in a hydraulic jump, and for larger discharges the jump was swept out and a jet was formed off the end sill of the basin.

I INTRODUCTION

General Description of the Dillon Project

Dillon Dam will be an earth dam which will span some 2900 feet across the valley of the Blue River about one-half mile downstream of the confluence of the Snake and Blue Rivers and Ten Mile Creek, near the town of Dillon, Colorado. See Figs. 1 and 2 for the location and vicinity maps. The storage reservoir created by the dam will be used to divert water through the Harold D. Roberts Tunnel for the municipal needs of the city of Denver. The reservoir will also be used to regulate the storage and diversion facilities of Denver's water system presently existing downstream of the Dillon Dam site.

The dam will be constructed in two stages. The first stage will be constructed to an elevation of 8896 ft. which represents a height of about 120 ft. above the present stream bed. The second stage will be raised to an elevation of 9031 ft. The spillway for the first stage will be a side channel spillway with an open chute to convey the flow. A deflector bucket will be constructed at the end of the chute to direct the flow well out into the existing stream channel away from the toe of the dam. The spillway for the second stage will be a morning glory spillway with a 15 ft. diameter concrete lined tunnel to convey the flow. See Fig. 3 for the plan and profile. It is the second stage spillway which was the object of the hydraulic model study reported herein.

The design of the second stage spillway system is complex. The spillway tunnel is initially a diversion tunnel during the construction of the dam. After completion of the first stage of the dam, the tunnel is used as a reservoir outlet. The cost of constructing a separate tunnel for the morning glory spillway is large and considerable economy can be realized by combining the second stage spillway tunnel with the reservoir outlet. (Hereafter the reservoir outlet will be referred to as the outlet). Serious hydraulic problems are developed in the system by such an arrangement and although there have been a number of morning glory spillways constructed in the United States, at the time of this writing, only a few have similar arrangements to the proposed design. There is a conspicuous lack of information from which to design the entire spillway with any degree of confidence. Because the damages caused by an inadequate design could be expensive to repair or maintain, it was desirable to check the preliminary design for performance and to make such modifications as found necessary to be consistent with hydraulic adequacy.

Scope of the Investigation

The specific features of the morning glory spillway studied in the model involved:

(1) -- Determination of the shape of the excavation based upon its effects on the flow approaching the spillway.

(2) -- Study of the shape of the spillway crest with possible modifications as determined by pressures measured on the face.

(3) -- Study of a flow regulator to control curvilinear flow within the spillway.

(4) -- Ventilation within the shaft to reduce negative pressures.

(5) -- Negative pressures in the system and the associated problem of cavitation.

(6) -- Design of the spillway-outlet tunnel intersection.

(7) -- Design of the deflector bucket at the exit to the tunnel.

II PROTOTYPE STRUCTURE

The preliminary plan and profile of the second stage spillway is shown in Fig. 3. The morning glory spillway crest is located on the left abutment. There is a drop of about 245 feet in the vertical shaft and bend from the crest to the beginning of the horizontal tunnel. The horizontal tunnel is about 970 feet in length with a deflector bucket at the tunnel outlet.

Spillway Crest and Surrounding Excavation

The preliminary design of the morning glory structure is shown on Fig. 4. The external diameter of the crest was 56 feet with the top of the crest forming a circle 53.2 feet in diameter. The elevation of the crest is at 9017 ft. from which the spillway tapers downward for a vertical distance of 58.5 ft. to elevation 8958.5 ft. At this elevation, the diameter of the circular tunnel is 15 feet and constant for the remaining length of the tunnel.

From the experience of previous morning glory spillways and model tests, (1), (2), it was believed that a crescent shaped excavation with a point located symmetrically would direct the flow satisfactorily. No piers were intended in the preliminary design, although it was recognized that a vortex suppressor might be required. The elevation of the bottom of the excavation was set at 9011 ft. in the preliminary design. A plan of the excavation is shown in Fig. 15.

Spillway Tunnel and Junction

The vertical shaft of the spillway tunnel extends from below the throat of the spillway shaft at elevation 8958.5 ft. to elevation 8854.44 ft. A circular bend with a radius of 75 feet connects the vertical shaft with the horizontal tunnel. Immediately downstream from the vertical bend, the spillway tunnel intersects the outlet tunnel of the same diameter and at the same invert elevation. In the preliminary design it was proposed that a curved outlet tunnel be joined with the spillway to enable merging of flows and thus minimize impingement and resulting damage from the outlet flows on the wall of the tunnel. The preliminary design of the outlet works, shown in Fig. 5, includes a 30 inch hollow jet valve (with two 4.75 x 4.75 feet high pressure gates on either side). The maximum capacity of the outlet will be 3000 second-feet at reservoir elevation 8886 ft., which is the elevation of the crest of the first stage spillway. Although the second stage reservoir surface elevation will be higher, the design outlet discharge remains 3000 c.f.s.

The Deflector Bucket

A deflector bucket is a structure to direct the high velocity flow away from the outlet to provide an economical means to dissipate the kinetic energy of flow. It is not an energy dissipator in itself, for the energy is dissipated by impact, shear and turbulence in the downstream channel away from the main structure. Figure 6 shows the preliminary design of deflector bucket A as tested in the model. It was reasoned that by a gradual change in direction, loss in momentum of the flow would be small, therefore, the trajectory of the jet would extend farther downstream.

III THE MODEL

Analysis of various model scales showed that a 1:30 scale reproduction of the spillway and appurtenant works would be satisfactory within the available laboratory space and facilities. A smaller scale model could introduce difficulties in interpreting model results because of model effects, whereas, a larger scale model would have added little, if any, significant data and would have cost more to construct and operate.

A model scale ratio of 1:31.31 was used, which was sufficiently close to the scale of 1:30 to be satisfactory. The actual scale was predicated on the size of the plastic pipe available commercially, used to simulate the 15-foot diameter concrete lined tunnel of the prototype. The size of plastic pipe used was 6 in. O.D. with one-eighth inch thick walls, giving an inside diameter of 5.75 in. Because extruded plastic pipes are not always uniform in size, the diameter of each pipe length was checked before being used. A tolerance of plus or minus one percent of the normal diameter was allowed.

Design of the Model

The major problem in designing the model was in determining the proper length of tunnel to be used in the model. This problem was created because of the disproportionately higher wall roughness of the plastic pipe as compared to the wall roughness of the concrete lined prototype tunnel. To correct this condition in the model the slope of the tunnel may be increased, the length of the tunnel may be reduced, or a combination of both methods may be employed. For any method used, the procedure involves calculation of the prototype velocity at the tunnel outlet for a given discharge and determining the proper length and slope of the model tunnel for the same discharge to give simulated model velocities at the outlet. For the Dillon Model, decrease in horizontal length of the tunnel was the chosen method, with the same slope of the tunnel as in the prototype.

Computations of prototype and model velocities are given in Tables 1 and 2 respectively. Head losses for the spillway crest to the end of the vertical bend were assumed for both model and prototype. These losses totalled approximately 10 percent of the total head, measured from the reservoir surface to the bottom of the vertical bend. The total correction in the length of the tunnel for the model resulted in a reduction of 9 feet, or about 29 percent of the total length of the tunnel as determined by the linear scale ratio of the model.

Limits of the model are shown superposed on the prototype layout in Fig. 7.

Construction

A schematic drawing of the model is shown on Fig. 8, and a photographic view of the completed construction is shown in Fig. 9. The overall dimensions of the headbox were 12 feet long, 8 feet wide and 3 feet deep. The spillway was located advantageously within the head box to make the most effective use of the space available. The water entered the spillway area of the model from three sides through rock baffles, with a maximum average discharge of about 0.13 cubic feet per second through each square feet of baffle area. This value is satisfactory for model inflow conditions, and any flow characteristics at the spillway excavation and crest can then be properly attributed to the influence of the topography and not to model inflow conditions.

Figures 10 and 11 show the spillway crest under construction. Construction of the spillway crest was greatly facilitated by setting up a turn table with a central axis around which a half template was rotated. This method assured a circular crest without irregularities.

There were four lines of piezometers constructed on the crest to measure the pressures on the face of the crest, see Fig. 10. The position and orientation of the piezometers with respect to the topography and tunnel are shown in Fig. 68. Clear plastic tubing was attached to the piezometer leads and connected to a manometer board shown in Fig. 13. A shut-off arrangement was set up so that only two lines of piezometers could be read at one time on the manometer board. This reduced the number of manometers necessary to measure the pressures.

Topography of the spillway crest site was modelled in concrete from elevation 8975 ft. to 9031 ft. Because of the low model inlet velocities, it was unnecessary for the topography to extend to a great depth in the model. Allowances were made in the construction of the topography for modifications of the excavation around the spillway to be made without necessitating major reconstruction. See Fig. 12 for a photograph during construction. The elevation of the excavation could be lowered by 11 feet to elevation 9000 feet. The shape of the excavation could be changed with a minimum of time and effort, by casting removable blocks as shown in Fig. 16.

The circular bend of the spillway tunnel was the most difficult construction problem. An exterior mold of the bend was first constructed of reinforced plaster. The plastic pipe was then placed in the mold and heated with oil under pressure, to force the wall of the pipe to conform to the mold. Subsequent cooling of the oil with the pressure maintained in the pipe retained the desired shape. A stage in the construction of the mold is shown in Fig. 14.

Figure 8 shows the preliminary design of the junction of the outlet and spillway tunnels as constructed in the model. Modifications of the junction will be described elsewhere in this report under appropriate sections. The method used for bending the outlet tunnel was the same as that described above for the vertical bend.

The tailbox was constructed with overall dimensions of 25 feet long, 6 feet wide, and 3 feet deep, as shown in Fig. 8. Conformance to river-bed material in the model was made by placing 3/4 to 1 1/2 in. gravel in the downstream area. In order to avoid excessive erosion of the topography during the early stages of testing, the side walls of the excavated channel were constructed with plywood. This also facilitated measurement of the jet trajectory from the deflector bucket.

A movable gate was constructed at the downstream end of the tailbox to control the tailwater level. A stilling well was placed in the tailbox to measure tailwater depths. The overall view of the model in Fig. 9 shows the headbox in the background with the tailbox in the foreground.

Water Supply

Water was supplied to the model from two pumps. An 8-in. high head pump supplied water to the headbox, and a 4-in. centrifugal pump supplied water to the outlet works. The flows into the model were measured with orifices installed in the discharge lines of the pumps. Calibrations of the orifices were checked at the beginning of the investigation.

IV. MODEL INVESTIGATION

Excavation of the Spillway Approach Channel

Preliminary Excavation and Location of Crest -- The preliminary shape of excavation is shown on Fig. 15. The crescent outline of the cut merges to a point. The purpose of this shape was to direct the flow radially into the spillway from both sides. Because of the momentum of flow, however, the flow was not radial and a water fin was formed where the flows combined from both sides of the point and extended into the spillway. These fins can be seen in the photographs of Figs. 16 to 21. Figure 21 shows submergence of the crest for a discharge of 16,000 c.f.s. The rating curve for the spillway is shown on Figure 22.

Design discharge for the spillway was 11,000 c.f.s. with allowance for an additional 500 c.f.s. for maximum conditions. Because of the high negative pressures which developed in the vertical shaft at large discharges, and the spiral flow at the vertical bend of the spillway tunnel, it was considered desirable to introduce a deflector above the bend to regulate the pressures and to control the flow surface through the bend. Inasmuch as the deflector would control the total discharge through the spillway and thus alter the flow pattern, the size of the deflector was first determined. Conditions and test results of pressures, flow through the bend, and design of various deflectors will be discussed in subsequent sections of this report.

The recommended deflector size was 4.5 feet, located immediately above the bend. With this deflector, excavation studies were resumed. Figures 23 to 26 show the effect of the deflector on the flow through the spillway for discharges of 7,000, 9,000, 11,000, and 11,500 c.f.s. respectively. Compare Figs. 24 and 25 with Figs. 18 and 19. The primary difference is that the point of submergence occurs for a smaller discharge. This was expected since the deflector is an obstruction in the conduit which reduces the capacity of the vertical shaft to convey the flow. The rating curve of Fig. 22 with the deflector also reflects the difference. For a discharge slightly in excess of 11,500 c.f.s., the crest becomes submerged, under which condition the spillway flow is controlled as an orifice by the opening at the deflector. The discharge then varies as the one-half power of the head above the constriction rather than as the three-halves power of the head on the crest. This causes the sharp rise in the rating curve near a discharge of 11,500 c.f.s.

There was a slight oscillation of flow for a discharge of 10,000 c.f.s. when the top of the boil was about at the same elevation as the spillway crest. To improve upon this condition, and to eliminate the fin, an arrangement of six piers was tried as shown in Fig. 27. Results for discharges of 7,000, 9,000, 11,000, and 11,500 c.f.s. are shown in Fig. 28 to 31. From the photographs, it is evident that the flow is distributed more uniformly around the spillway crest and flow is radial. There is no local concentration of flow and generally good flow conditions prevail. Although the arrangement of 6 piers improve flow conditions at the crest, the reservoir water surface was raised because of the reduced crest length as shown in Fig. 21. Therefore, it was desirable to attempt similar improvements by altering the excavation of the approach channel.

First Modification - Nonsymmetrical Excavation -- Observation of Figs. 23 to 26 shows that although the point of submergence is at a smaller discharge with the addition of a deflector in the vertical shaft, the pattern of flow in the approach channel is not visibly altered. Therefore, change in the flow pattern can only be controlled by change in excavation.

An unsymmetrical excavation was tested with the dimensions shown on Fig. 15. The intended purpose of this plan was to attempt better distribution of flow along the crest and to study the effect of excavation on the flow pattern. It was also intended that the fin be directed against the deflector in the vertical shaft and, thus, control the flow surface in the horizontal tunnel. The effects of this excavation on the flow pattern for discharges of 7,000, 9,000, 11,000, and 11,500 c.f.s. are shown in Figs. 32 to 35.

The excavation was unsatisfactory because spiral flow was created in the vertical shaft and a large portion of the flow by-passed the deflector, and consequently set up undulating flow in the tunnel. Also, at discharges of 11,000 and 11,500 c.f.s. a vortex was created which reduced the capacity of the spillway. The reservoir surface for a discharge of 11,500 c.f.s. was much higher than the allowable water surface. Figs. 36 and 37 show flow in the vertical shaft and at the deflector for discharges of 9,000 and 11,500 c.f.s. The spiral flow in Fig. 36 can be seen by the trace of air bubbles. In Fig. 37 a vortex pocket is evident a short distance below the floor of the headbox. This vortex pocket was very unstable with a constant change in size and location. Because of this, the pressures measured in the vicinity of the air pocket fluctuated considerably.

Second Modification - Over-Excavation at El. 9,011 feet -- To determine if improved flow conditions could be realized by extending the limit of excavation laterally, the removable blocks at El. 9,011 ft. were taken out. This resulted in a rectangular shaped cut shown in Fig. 15. Figures 38 to 41 show flow patterns at discharges of 7,000, 9,000, and 11,000 c.f.s. Figures 40 and 41 show that for a discharge of 11,000 c.f.s. the crest was submerged and the reservoir surface level was increased because of the development of a vortex.

When piers were installed on the crest in the same arrangement and number as shown in Fig. 27, the vortex was suppressed which enabled the spillway to discharge the design and maximum capacities. The effects of the piers can be seen in Figs. 42 to 45.

Third Modification - Bottom of Cut lowered to El. 9,000 ft. -- The excavation of the second modification was lowered to 9,000 ft. Lowering of the cut improved the coefficient of discharge, and because the reservoir surface did not rise to submerge the crest, a vortex was not formed. Figures 46 to 49 reflect this improvement for discharges of 7,000, 9,000, 11,000, and 11,500 c.f.s. The improvement was significant, but the extent of excavation was excessive; therefore, a narrower excavation was tried as the fourth modification.

Fourth Modification - Closer Limits of Excavation at El. 9,000 ft. -- The limit of excavation was brought to closer proximity of the crest. The dimensions are shown in Fig. 15. Flow conditions are presented in Figs. 50 to 53. Figure 53 shows a vortex about to be formed for discharge of 11,500 c.f.s. Otherwise, the flow pattern is satisfactory. Note that as the velocity of approach is increased, a fin is again formed because of the momentum of flow from opposite sides of the crest.

In summary of the excavations studied to this stage, although excavation to elevation 9,011 ft. is satisfactory; improvement in the coefficient of discharge can be realized by lowering the base of the cut to elevation 9,000 ft. Determination of an optimum elevation would require studies at several intermediate elevations. The lateral limits of the excavation should be extended from the fourth modification, and again an optimum must be found by experimentation. However, because there will be some effects of approach velocity on the flow at the crest, a fin cannot be prevented from forming.

Fifth Modification - Vertical Cutoff Wall on Crest -- A shaft Spillway with a cutoff wall as shown on Fig. 15 was tried in an attempt to eliminate the fin and to achieve radial flow at all points on the crest. This type of wall would eliminate need for approach excavation entirely. Results of the study indicated that good flow conditions were obtained for all discharges. Slight oscillation of the boil existed for a discharge of 10,000 c.f.s. Although dynamic forces were not measured, there could be danger of vibration of the structure. Because a considerable length of the crest was made ineffective by the cutoff wall, the water level was raised in the reservoir. If a crest of this design was to be used, the radius of the crest would have to be increased to attain the same crest length as the circular spillway. This would result in a larger spillway crest structure and a longer tapered throat into the vertical shaft.

At this stage of the study, the crest structure was changed in location by the designers. New excavation plans were therefore studied to fit the relocated spillway.

Sixth Modification - Relocated Spillway Crest -- The plan of excavation is shown on Fig. 54. Because the spillway crest with the cutoff wall performed staisfactorily for the preliminary location, in so far as the approach flow was concerned; the same crest was first investigated for the relocated crest. The same condition of flow was not achieved however, for the relocated spillway. There was pile-up of flow along the vertical wall because of the angular component of flow. With a cutoff wall, a larger radius crest would be required which would place part of the spillway structure back on the undesirable foundation. It was concluded, therefore, that because there was no improvement in the flow condition at the relocated site from a circular spillway, a morning glory spillway would be used. Thus, the crest with a cut-off wall was abandoned from further study.

Seventh Modification - Relocated Spillway Crest -- A full morning glory crest was subsequently tried with a crescent shaped cut with a point located symmetrically to control the flow which merged from the two sides. The dimensions of this excavation are given in Fig. 54. Excavation at elevation 9 000 ft. was studied initially and compared to excavation at elevation 9011 ft. With the additional channeling necessary at the relocated site, it was desirable to keep the elevation of the cut as high as possible. The rating curve from Fig. 55 shows that the coefficient is improved for the lower cut, a significant effect is not in evidence. Therefore, the excavation of the channel was raised to elevation 9011 ft.

Eighth and Ninth Modifications- Final Spillway Locations -- These excavation studies were made for the spillway crest which was changed in location again by the designers. The initial excavation for this location of the crest is shown in Fig. 56. Which is essentially the same as that shown in Fig. 54 but with the extension of the extremeties of the cut.

The approach channel was well defined and cross-flow developed on the crest. Radial flow existed across only a small portion of the crest. As a consequence, the crest was less efficient, and a higher stage-discharge curve resulted as shown in Fig. 57.

To check the dependency of the flow direction at the crest on the geometry of the excavation a test was made with the control point partially removed. With the reduced control point, a vortex developed at the crest which reduced the efficiency of the spillway sufficiently that the design discharge would not be conveyed through the shaft spillway without overtopping the dam as shown in Fig. 58.

In view of the serious consequences suffered if the point of excavation was removed, it was considered desirable to either lower the excavation to elevation 9000 ft. based on the result of tests with excavation No. 7 or to install piers on the crest as a measure to control the vortex. From hydraulic considerations, piers were more desirable. Piers would not only control the vortex, but distribute the flow more uniformly around the crest, while lowering the bed of excavation would not effect a major improvement in the distribution of flow. Figures 59 and 60 show the effects of a 4 pier and 6 pier arrangement on the crest. The improvement in the flow for a discharge of 11,500 c.f.s. by 6 piers is not discernable over 4 piers. Also, use of 6 piers reduces the crest length by an additional width of 2 piers over 4 piers and, thereby, causes an increase in the water surface level of the reservoir. It was concluded, therefore, that 4 piers would be used on the crest.

The piers were rotated in a number of different positions and the most satisfactory arrangement was found to be that shown in Fig. 54 with the piers located at 45^0 to the centerline of the tunnel. More effective use of the piers resulted and also a better distribution of flow was effected around the crest. Figures 61 to 64 show the flow condition with the recommended location of piers at discharges of 3,000, 7,000, 9,000, and 11,000 c.f.s. Flow patterns made by traces of confetti are shown for discharges of 7,000 and 11,000 c.f.s. in Figs. 65 and 66. The rating curve for the recommended condition is shown in Fig. 67.

Spillway Crest Pressures

Pressures on the face of the morning glory spillway, throat and vertical shaft were measured to determine development of excessive negative pressures. The location of piezometers is shown in Fig. 68. In all, there were 4 lines of piezometers with 18 piezometers in each line for a total of 72. The piezometers were connected to the manometer board shown in Fig. 13 in such a way that two lines of pressures could be read and recorded at one time. By using a wye connection at the bottom of the manometers and a simple clamp device, the other two lines of pressure could also be read by the same set of manometers.

The four piezometer lines were oriented 90 degrees to each other and 45^0 to the centerline of the tunnel as shown in Fig. 68. This arrangement enabled determination of unusual pressures peculiar to the location. The accuracy of pressure measurement is about $\pm$ 0.5 ft. of water in prototype dimensions. Effects of surface tension in the manometer tube and accuracy of reading the water surface level contribute to the errors.

Pressures on the preliminary crest for various discharges are tabulated in Table 3. Figure 69 shows the pressures measured at piezometer line B, for discharges of 3,000, 7,000, 9,000, 10,000 and 11,000 c.f.s. Pressures for discharges to 9,000 c.f.s. were acceptable. However, for discharges larger than 10,000 c.f.s. the negative pressures in the vertical shaft and throat of the crest were excessive. Prototype values to vapor pressure were measured in the model.

There are several methods by which negative pressures on the crest and throat can be reduced. First, the crest diameter can be enlarged and the change in slope of the crest made more gradual so that positive pressures are developed everywhere on the crest. This would result in a larger and longer crest.

The second method is to construct a deflector in the vertical shaft to develop positive head above the constriction for high discharges. The problem created by this method is that the discharge through the spillway is limited by the size of the opening at the deflector, thus a condition of maximum allowable negative pressure and maximum spillway discharge required would not necessarily be compatible.

The third method for alleviating negative pressures is to provide air vents through the zone of high negative pressures. A model study of the Hungry Horse Spillway by the Bureau of Reclamation, (1), showed, however, that this method is local in effect and a complex system of ventilation would probably be needed to make the system effective.

Of the three methods, the second was considered the most feasible, and structurally satisfactory. Hydraulically the deflector would perform an added function of straightening the flow through the bend.

A study was made therefore, to determine if a deflector would solve the problem of excessive negative pressures and then to determine the size of deflector required. A maximum discharge of 11,500 c.f.s. was required to pass through the spillway at a reservoir water surface elevation not to exceed 9025 ft. with a maximum allowable negative pressure in the shaft of 11 feet of water.

Figure 70 shows the various sizes and types of deflectors studied. The pressures measured for the deflectors are tabulated in Table 4. The effect of the various deflector sizes on the rating curve of the spillway is shown in Fig. 71. As the deflectors increase in size, quite obviously the maximum spillway discharge for a given reservoir elevation reduces proportionately. To show the effect of deflector size on pressures in the spillway, curves of measured maximum negative pressures were drawn as a function of deflector size and discharge. See Fig. 72. Deflectors 1 through 5 only were used in this study. From this curve, it was observed that a deflector size of 4.5 ft. would allow a maximum negative pressure of 11 feet for a discharge of about 11,500 c.f.s. A check with the rating curve of Fig. 71 showed that a discharge of 11,500 c.f.s. could be safely conveyed through the spillway. Thus, a 4.5-foot deflector was installed in the model. In the tests described above the deflector was located immediately above the vertical bend with the bottom of the deflector at the point of curvature of the bend. Ventilation to the atmosphere was provided directly below the deflector. To ensure sufficient model ventilation three-3/4" holes were made in the pipe wall.

A tabulation of pressures for the 4.5 foot deflector is given in Table 5 with maximum negative pressures plotted in Fig. 73. The maximum negative pressure was within the allowable limit. The preliminary crest shape was therefore, considered satisfactory and it was not necessary to test different shapes of the crest.

A limited study was made to determine if other shapes of deflectors would be more functional. Fig. 70 shows a crescent shaped and a dentated deflector with about the same constriction as a 4.5 foot deflector with a straight edge. For large discharges of flow through a crescent deflector it resulted in undesirable flow conditions through the bend. as the flow passed the deflector and intersected the wall of the circular bend, a fin was formed which flowed over the crown of the tunnel through the lower portion of the bend and caused an undulating water surface in the tunnel. The dentated deflector was not a functional improvement over the straight edged deflector. The flow was seriated at the surface which tended to cause irregularity in the surface of flow. Although no measurements were made, visual observation indicated additional air entrainment in the flow because of the increased turbulence at the air-water interface. The centrifugal force of the flow in the bend caused the flow to attain a level surface transversely, and in so doing, any disturbance of the water surface caused at the deflector become a source of surface undulation downstream in the bend which is propagated into the tunnel.

Some pressures were measured on the crest and in the vertical shaft of the spillway with the two deflectors, but are not included in this report. Pressures upstream of the deflector cannot be expected to be materially different from the straight edged deflector. From consideration of fluid mechanics, it can be shown that the flow lines, and hence the potential lines upstream from the deflector in the zone of maximum negative pressures cannot be materially different for any of the deflector shapes.

A limited study was also made for different locations of the deflector in the vertical shaft. Although deflectors are usually placed at the bend, the effect on the spillway capacity and the flow for other locations was studied. Tests of the 1-3/4" deflector at 40 and 80 feet above the bend showed that the higher the deflector is situated in the vertical shaft, the less will be the maximum spillway capacity for a given deflector size and reservoir elevation. Also, as the deflector is raised above the bend and the distance from the deflector to the bend is increased, the surface of the flow is disturbed and smooth flow does not occur through the bend. When smooth flow is not attained through the bend, undulation of the water surface in the tunnel results. The spillway rating curves with the deflector at different location are shown in Fig. 74.

Spillway Outlet Tunnel Junction

The Preliminary Design - The preliminary design of the tunnel junction was developed in the laboratory. The dimensions are shown in Fig. 8. The general plan was to form a curved tunnel downstream from the gate to merge the flows smoothly with the spillway flows. A weir was constructed across the opening as a formed invert for the high velocity spillway flows, to reduce spreading at the junction and prevent spiralling flow downstream from the junction. The undulation of the surface developed by the spiral could cause nonsymmetrical flow at the deflector bucket and form a jet which would concentrate in one side of the excavated channel.

Tests conducted with the preliminary design showed that flows through the spillway tunnel were satisfactory with the formed invert. However, for flows through the outlet with no flow through the spillway, the high velocity flow from the valves, unable to turn according to the radius of the curve, flowed up the wall of the circular tunnel. This flow condition occurred for discharges from 1,000 to 3,000 c.f.s. The discharge of 3,000 c.f.s. is the maximum design outlet flow. For discharges from 500 to 1,000 c.f.s. the water flowed along the invert of the conduit with sufficiently great velocity to ride over the weir. The energy of flow thus dissipated by impact on the weir, caused a hydraulic jump to form in the downstream section of the horizontal tunnel. The location of the jump varied with the discharge. For a discharge of 500 c.f.s., the jump formed near the weir, while for a discharge of 1,000 c.f.s. the jump formed near the tunnel outlet. For discharges less than 500 c.f.s., the wier provided sufficient obstruction to cause a hydraulic jump to form in the outlet tunnel. For the large discharges through the outlet valves above 1,000 c.f.s., the flow imparted considerable force on the wall of the spillway tunnel at the junction. In the prototype, it was likely that protection from erosive forces would be required in the spillway tunnel for the length of the junction.

Combined spillway and outlet discharges were also tested. Spillway discharge ranged from 3,000 to 11,500 c.f.s. and the outlet discharge from 0 to 3,000 c.f.s. For discharges in the spillway tunnel less than 7,000 c.f.s. and in the outlet tunnel up to 3,000 c.f.s., no peculiar flow conditions were observed. Turbulence at the junction resulted in air entrainment but caused no difficulty of flow in the tunnel.

When the spillway discharge was increased to 9,000 c.f.s. and with an outlet discharge of 2,000 c.f.s., the outlet tunnel flowed nearly full. As the outlet discharge was increased to 3,000 c.f.s., the tunnel flowed full and some back pressure was developed at the gates. Back pressures were intensified as the spillway discharge was increased further. It was evident that with back pressure at the gates, a hollow jet valve could not be used in the outlet works. The hollow jet valve was subsequently replaced with a high pressure gate, and the outlet works was redesigned to include two 4' x 5' gates centered by a 2.25' x 2.25' square high pressure gate.

A filled invert was constructed in the tunnel from the gates to the weir to a height equal to that of the weir. The filled invert eliminated the hydraulic jump in the outlet works tunnel and reduced the energy dissipation at the weir. As a result, a hydraulic jump was not formed in the tunnel for discharges greater than about 600 c.f.s.

The filled invert, however, did not significantly improve the flow characteristics. It was observed that the spillway tunnel still flowed nearly full for a combined discharge of 12,000 c.f.s., largely because of the air entrainment in the flow from the turbulence developed at the junction. In the prototype the comparable discharge will be somewhat less than 12,000 c.f.s., because the amount of air entrainment would be more than that in the model. At the same time for full pipe flow, frictional resistance is greater in the model than that of the prototype. The end result is that prototype discharges can be expected to be less than 12,000 c.f.s. because frictional resistance has been partially compensated for by reduced model tunnel length.

Straight Outlet Tunnel Junction -- The curved outlet tunnel was unsatisfactory. A straight tunnel which formed a junction with the spillway tunnel at an angle of 30^0 was subsequently tested. A plan of the junction is shown in Fig. 75(a). Figures 76 to 85 show various flow conditions at the junction for a number of spillway-outlet discharge combinations. This design of the junction was not an improvement over the preliminary design. There was as much, or more, impact against the tunnel wall and turbulence and air entrainment in the flow at the junction.

Discharges less than 500 c.f.s. caused no material problem, but as the discharge increased to 1,000 c.f.s., a spiraling flow developed in the spillway tunnel as shown in Fig. 79. As the flow was further increased to 3,000 c.f.s., considerable impact on the spillway tunnel and spiral flow was evident. See Fig. 81.

For combined flows, the flow characteristics were similar to the preliminary design.

Junction at the Vertical Bend -- A horizontal spillway-outlet junction, either with a curved outlet tunnel as in the preliminary design, or a straight tunnel at 30^0 of the modified design was unsatisfactory. The impending forces on the tunnel walls were conducive to extensive damage. To reduce the forces, the flow from the valves could be dissipated before the junction, or the flow from the outlet could be aligned with the spillway tunnel. It was not considered feasible to dissipate the large quantity of flow from the outlet works in the tunnel. Thus, the outlet was realigned with the spillway tunnel. Two designs of the ensuing junction at the vertical bend of the spillway shaft were made and tested. Because the outlet works were to be constructed in the diversion tunnel, the inverts of the outlet and spillway tunnels at the junction must be at the same elevation. The two designs were different only in the size of the outlet conduit. One was 8 ft. high by 15 ft. wide, while the other was 8 ft. square in cross-section. Both are shown in Fig. 75(a).

Some of the tests conducted with the 8 x 15 foot outlet conduit are shown in the photographs of Figs. 86 to 89. With no spillway flow and outlet works discharge only, satisfactory conditions prevailed at the junction. There was air demand in the vent immediately downstream of the gates; greater demand for larger discharges. The surface of flow was relatively smooth through the entire tunnel. No area of impact was observed. Table 6 gives the pressures measured at the junction. The location of piezometers is shown in Fig. 75(b).

Tests conducted with flow from the spillway only, showed that the abrupt change in conduit section at the junction caused flow to impinge heavily on the bottom of the rectangular section. The ensuing flow spilled over the crown of the tunnel causing the tunnel to flow full for about four to five feet downstream from the end of the vertical bend. Considerable air was entrained in the flow accompanied by air demand from the vents below the deflector.

Combined discharges from the outlet and spillway did not improve the tunnel flow from the condition just described. A photograph of the combined flow is shown in Fig. 88. Compare with Fig. 89.

Recommended Junction at the Vertical Bend -- Subsequent to the 8 x 15 ft. conduit junction at the bend, an 8 x 8 ft. conduit was constructed and tested in the model. The 8 x 8 ft. junction performed much more satisfactorily, primarily because of better continuity of the boundary at the bottom of the vertical bend. Figures 90 and 91 show flow through the outlet works only, for discharges of 1,000 and 3,000 c.f.s. Although a fin is formed downstream of the gates because of the transition to an 8 x 8 ft. conduit from the gate section, no adverse effects were evidenced. As is shown in Fig. 92, a flow of 3,000 c.f.s. through the spillway with no flow in the outlet caused a water fin to form in the horizontal tunnel. The tunnel was not closed off, however, the fin is reduced as the outlet flow is merged with the spillway flow. Figure 93 shows this improvement for a combination of 3,000 c.f.s. through both the spillway and the outlet.

Photographic comparisons show the differences in the two junction designs. In Fig. 88 for a discharge of 7,000 c.f.s. through the spillway and 2,000 c.f.s. through the outlet, the tunnel is closed off, while in Fig. 95 for a spillway discharge of 7,000 c.f.s. and outlet discharge of 3,000 c.f.s. the tunnel is not closed by the flow. Table 7 gives pressures at the junction for a wide range and combination of flows before the junction was vented and Table 8 gives the pressures after vents were installed. For combined flows, air was necessary through both the deflector vents and the gate vent. As the spillway flow increased, the centrifugal force of the flow through the bend caused pressures to be developed at the gates. Back pressures were not great enough to retard the outlet flows however.

There was considerable air entrainment in the model flow. The ratio of air flow to water flow in the prototype will be greater than that of the model with the result that less water capacity can be expected in the prototype than that indicated by the model for a given energy head. The difference between the model and prototype will be the difference in the amount of air entrained.

Several minor modifications were subsequently tried at the junction but no material improvements were noted. One modification involved a small deflector in the bend above the junction with the conduit. This deflector was intended to direct the flow outward to decrease the impact of the flow on the bottom of the rectangular conduit. However, standing waves were formed because of the sudden change in flow direction which resulted in spillover at the crown of the tunnel. The deflector was removed and an extension of the vertical tunnel into the square conduit opening was tried with the intent to reduce the angle of impact. This also proved unsatisfactory as flow from the outlet works caused dynamic forces on the extended lip subjecting it to probable damage and maintenance problems. Because the flow area was reduced, greater back pressure was also created downstream from the gates.

The junction as shown in Fig. 75(a) was, therefore, recommended for construction.

Air Vents

Air vents in the vertical bend below the deflector have previously been discussed. Determination of the size of vents required in the prototype was made from a qualitative study. Under the most demanding conditions, an air duct of about 36" in diameter will be needed to supply air to the spillway tunnel below the deflector. For the recommended junction of the spillway and outlet tunnels an air vent system as shown in Figs. 98 and 99 will be required. In addition to the air downstream from the gates, air will be required at the end of the transition and also at the intersection of the inverts of the vertical bend and the square conduit. In order to avoid recirculation of water through the air vents, separate conduits to the ventilation points would be preferable. Each of the air ducts should be at least 12 in. in diameter and designed to withstand hydrostatic pressures of about 150 ft. of water. A duct of about 24 in. in diameter should be supplied to the gates and preferably a manifold constructed to provide air from at least two inlets at the top of the conduit as close to the gates as practicable.

Deflector Bucket and Stilling Basin

Deflector Bucket A -- The preliminary design of the deflector bucket at the tunnel outlet was set as a cubical curve with an angle of about 10^{0} at the lip of the bucket. See Deflector Bucket A in Fig. 6 for the dimensions. The term "deflector bucket" is not to be confused with the term "deflector" as applied to the structure in the vertical shaft. In tests with this bucket, a well formed jet did not develop. The small angle at the lip of the bucket was not sufficient to arch the trajectory. It was also noted that a long structure was unnecessary. A circular deflector bucket with a small radius at the exit would probably function adequately, and a transition structure would not be needed in developing the jet trajectory.

Deflector Bucket B -- The dimensions of this structure is shown in Fig. 100. Because of the small angle at the lip, however, this design was not an improvement over the preliminary design. The form of the trajectory was very similar to the first because of the small angle and height of the lip of the deflector bucket. For discharges less than 5,000 c.f.s., the flow did not separate entirely from the end of the structure, and a well-formed jet did not develop. For discharges greater than 5,000 c.f.s., the velocity was sufficiently large to overcome this tendency and a jet was formed.

Deflector Bucket C -- In view of the difficulty of attaining a satisfactory jet trajectory with a small exit angle and height of the lip, the angle was increased to 20^0 and the height to 7 ft. Although in this design the structure formed an improved jet from the previous ones, there appeared to be insufficient height to the jet trajectory. The low trajectory, particularly for high discharges, would scour a considerable length of the bed and banks of the downstream channel. It seemed desirable, therefore, to develop a jet with less horizontal component of velocity at the end of the trajectory.

Deflector Bucket D -- This design was intended to increase the height of the trajectory and to overcome the deficiencies of Design C. See Fig. 100 for the dimensions. The lip angle was increased to 30^0 while the height was maintained at 7 feet. An improvement in the jet trajectory was noted, but further studies were desired to determine the optimum angle.

Deflector Bucket E. -- A change in the shape of the deflector bucket was studied in this design. The main purpose of this deflector bucket shape was to eliminate the standing wave at the sides of the structure due to the abrupt change in flow boundaries from a circular tunnel to a rectangular section. From Design D it was noted that an angle of 30^0 was an improvement over Design C. A change in the shape of the structure, and a slight change in the height of the lip made it desirable for comparative purposes to make another study with an angle of 30^0. The drawing of this structure is shown in Fig. 101.

Results of tests made with this structure showed that there was some improvement in the shape of the jet. Although a standing wave was still evident at the lip of the deflector bucket, this was due largely to the concave surface developed by the centrifugal force of the flow on the circular curve of the bucket. As the quantity of flow is concentrated near the center of the structure because of the circular tunnel, the flat bottom of the deflector bucket tends to force the flow to the sides of the structure. The result is a non-level water surface at the lip and a smooth jet is not formed. The discrepancy from a theoretical jet however, is not significant in effect.

Deflector Bucket F -- To check the effect of a greater angle at the lip, Design F was increased to 35^0. The drawing is shown on Fig. 101. The angle resulted in a higher trajectory than Design E, and better conditions were evident.

Deflector Bucket G -- A lip angle of 40^0 was tested for comparison with Designs E and F. The drawing is also shown on Fig. 101. The angle was too large, and the jet in developing a higher arch lost momentum and thereby resulted in greater spreading and a wider impact area on the channel bed. A considerable amount of the sides of the channel would have been eroded. In comparing visual results of the 7 deflector buckets it was concluded that Design F produced the most satisfactory jet trajectory. See Fig. 102 to Fig. 105 for photographs of jet trajectories of deflector bucket F.

All of the deflector buckets, possessed one undesirable condition which is inherent in its design. At small discharges of from 0 to 1,000 c.f.s., depending on the lip height of the bucket, the flow does not possess sufficient velocity and momentum to form a jet. A hydraulic jump is therefore formed in the tunnel which varies in location with the discharge. The subcritical flow beyond the jump in the tunnel discharges through critical depth over the end sill. See Fig. 102 for a representative photograph. This flow condition, which would exist for the daily outlet flows of 200 to 500 c.f.s. in the prototype, would develop a scour hole that would undermine the structure. A protective slab or a pool adjacent to the end sill could be constructed, but this would not solve the problem. At some discharge the jet would impinge just beyond the paving or the wall of the pool and develop a scour hole. Rapid failure of the entire structure would then result. It appeared, therefore, that a deflector bucket by itself was not an adequate structure at the outlet of the tunnel to accomodate the entire range of flows expected.

Stilling Basin H -- From previous studies, it was recognized that a stilling basin would be required. The length of stilling basin required to adequately dissipate the energy by a hydraulic jump for a discharge of 11,000 c.f.s. would be excessive and unjustifiable from a hydraulic standpoint. Therefore, it was decided to develop a combination structure, where discharges to 3,000 c.f.s. would be contained by a jump, but flows greater than 3,000 c.f.s. would sweep through the structure and form a jet off the end sill of the basin. Dissipation of energy would be effected in the channel some distance from the structure. The significance of

3,000 c.f.s. as a limit set for the hydraulic jump in the stilling basin was that this quantity was the maximum design flow through the outlet works. Discharges normally expected from the outlet should always be contained as a hydraulic jump in the stilling basin.

Dimensions for the initial design of this combination structure is shown in Fig. 106. The entire structure was 130 feet long and 15 feet wide. A transition section of 62 feet in length was followed by the stilling basin 68 feet in length. The floor of the basin was at an elevation 7 feet lower than the inverts of the tunnel to provide adequate depth after the jump. Note also that the elevation of the tunnel was 6 feet higher than the previous invert elevation which resulted by a change made by the designers. No change was made in the model, however, as the difference amounted to about 2 percent of the total height. Also, in view of the necessary approximations used in determining model dimensions of the tunnel previously described, the difference of 2 percent is not significant.

The basin was tested for discharges of 1,000, 2, 000, and 3,000 c.f.s. through the outlet works as well as for discharges of 3,000, 5,000, 7,000, 9,000, and 11,000 c.f.s. through the spillway. A combination of spillway and outlet flows ranging from 4,000 to 14,000 c.f.s. was also tested. Observations and notes were taken of the flow conditions.

For flows of 1,000 and 2,000 c.f.s. the stilling basin contained the hydraulic jump. However, at 3,000 c.f.s. no jump was formed. For discharges through the spillway greater than 3,000 c.f.s., the flow swept through the basin and a jet was formed off the end sill. The shapes of the jets were unsatisfactory in that a high central "fin" was formed and the flow sprayed considerably on both sides of the downstream channel. It was evident that a 2:1 straight sill was undesirable as a deflector bucket.

A curved circular end sill was subsequently installed with some improvement in the jet flow. The major difficulty encountered in the formation of a jet was that the impact of the water on the floor of the stilling basin spread the flow at the point of impact and caused a standing wave at the sides of the basin. As the flow progressed downstream, the waves were propogated to the center and ultimately merged as a fin at the end sill. Figs. 107 to 110 show flows for discharges of 5,000 and 9,000 c.f.s.

Stilling Basin I -- A shorter overall structure was then tested. Dimensions of the structure are also shown in Fig. 106. The transition section was 30 feet long and the basin length 70 feet with a circular end sill and a lip angle of 35^0. The width of the structure was increased 7 feet to 22 feet.

The entire range of flow was tested in this basin. Observation showed that the transition portion of the structure was too short for the larger discharges. Flow from the tunnel outlet separated from the floor of the structure and impinged with considerable force on the curved end sill. The resultant jet from the end sill was, therefore, entirely unsatisfactory. Discharges of about 2,000 c.f.s. were contained in the stilling basin as a hydraulic jump. At 3,000 c.f.s., however, the jump was not formed.

Stilling Basin J -- The transition section of the structure was increased to 60 feet in length and the stilling basin length to 80 feet. The entire structure was constructed 22 feet wide. The stilling basin floor was lowered 2 feet to elevation 8767 ft.

Tests for discharges of 1,000, 2,000, and 3,000 c.f.s. through the outlet works were conducted. The discharge of 3,000 c.f.s. was not contained in the stilling basin. A deeper stilling basin appeared necessary. Pressures on the floor of the transition are given in Table 9.

Stilling Basin K -- The dimension of the stilling basin was the same as for Basin J., excepting the stilling basin floor was lowered to elevation 8763.5 ft. See Fig. 111. This basin was successful in containing a discharge of 3,000 c.f.s. through the outlet works as a hydraulic jump. The floor elevation was raised to elevation 8765 ft., but at this elevation the hydraulic jump was not stable. The stilling basin floor was, therefore, set at 8763.5 ft.

A test of the full range of discharge was then made. Table 10 gives pressures measured on the transition of the stilling basin with the divider wall. Experimentation with this stilling basin showed that better flow conditions resulted by installation of a divider wall on the transition section. This divider wall forced the formation of the standing wave farther upstream in the stilling basin, yielding a better formed jet from the end sill for discharges greater than 5,000 c.f.s. For a discharge of 5,000 c.f.s., an irregular jet was formed and flows less than 3,000 c.f.s. were contained in the basin as a hydraulic jump. The results are shown photographically in Figs. 112 to 118, and the water surface profiles are tabulated in Tables 11 and 12. Hydraulic jumps were contained in the stilling basin for discharges to 3,400 c.f.s. through the outlet works for increasing discharges without being swept out. Once the jump was swept out, it did not reform in the basin on decreasing discharge until a flow of about 2,200 c.f.s. was reached. For flows through the spillway, these limits were about 3,200 and 2,100 c.f.s. respectively. In translating these results to the prototype, however, it must be recognized that the jump will be swept out and reformed in the basin at discharges less than that indicated because of the differences in model and prototype boundary friction losses.

Scour tests were conducted in the downstream channel. The results are qualitative only and because no prototype data were available it was assumed for purposes of this study that the channel material was erodible. Graded gravel from 3/4 to 1-1/4 in. with a median size of about 1 in. was placed in the model. Results of 5 minute scour tests for discharges of 7,000, 9,000, and 11,000 c.f.s. are shown in Figs. 120 to 122 respectively. The location of the scour hole ranges from 100 to 160 feet with increasing discharge, measured from the end sill. The material adjacent to the structure was not noticeably eroded. The period of test, however, was very small.

Test with the same gravel material was conducted for a discharge of 3,000 c.f.s. with a hydraulic jump in the stilling basin. After 30 minutes of testing no noticeable scour resulted. The bed material in the model was subsequently changed to 3/16-in. median size sand. Results of 30-minute tests with discharges of 1,000, 2,000 and 3,000 c.f.s. with a hydraulic jump are shown photographically in Figs. 123 to 125. These tests indicate that the channel downstream from the stilling basin should be protected with rip rap. The rip rap should be graded and, in view of the model results, the largest individual pieces should be about 3 feet in diameter. It is recognized that the rip rap will provide protection of the channel for normal reservoir outlet discharges only. For flood conditions, and no hydraulic jump to dissipate the energy, some maintenance of the channel should be expected.

<u>Stilling Basin L</u> -- A trapezoidal stilling basin was tested to determine if hydraulic advantages were effected from a rectangular shaped basin. The basin however, was unsatisfactory because as the end sill increased in elevation it also widened. This contributed to an expanding jet and water was spread onto the channel walls. This would have required a wider channel downstream from the stilling basin to avoid damage. Stilling Basin K was subsequently recommended for construction.

SUMMARY OF THE RECOMMENDED DESIGN.

The final spillway design is shown in the specification drawings, part of which is included in this report as Figs. 98 and 99, and Figs. 126 to 129. The approach channel was excavated to elevation 9,011 ft., and was symmetrical about a line normal to the centerline of the tunnel. Four piers were located on the crest to control the angular flow and to effect better distribution of flow around the crest. A deflector 4.5 ft. wide was located on the vertical shaft of the spillway, at the circular bend to prevent excessive negative pressures from developing in the throat of the spillway and to control the flow surface through the bend and horizontal tunnel.

The recommended location of the deflector was at the point of curvature of the vertical bend. This elevation, throughout the tests, was at 8854.59 ft. In the final drawings, however, the bend begins at elevation 8866.32 ft., 11.73 ft. higher than the recommended condition. A review of the model results for tests of different deflector locations showed that only a minor difference in spillway rating curves would result. Pressures in the throat would not be measurably affected and it would be towards greater positive pressures. A check of the rating curve was made in the model for the final deflector location and it was found that for the maximum discharge of 11,500 c.f.s. the reservoir elevation would be 9 025.3 ft. For 11,000 c.f.s. the reservoir elevation was 9024.7 ft. For discharges less than 10,000 c.f.s. no change occurred. The final location and size of deflector is considered hydraulically satisfactory.

The reservoir outlet-spillway tunnel junction was developed at the location of the vertical bend. An 8 ft. x 8 ft. square outlet conduit joins the 15 ft. diameter circular tunnel at the bottom of the vertical bend. Air vents were provided at critical points in the junction to alleviate negative pressures and thus avoid cavitation.

The stilling basin at the tunnel portal was a combination hydraulic jump basin and deflector bucket. The final drawings show the transition section to the basin as a box conduit with a level top. This will not affect the hydraulics of the flow in this or succeeding sections since there is adequate space provided around the flowing water to allow air passage.

The horizontal tunnel in the final design shows a slope of 0.00845 as compared to a slope of 0.002 of the preliminary design. In the model, the slope was 0.002. The greater slope of the prototype tunnel will not materially affect the results as obtained in the model since the effective increase in velocities at the tunnel portal due to the slope is about 2 percent, assuming the same roughness coefficient.

The downstream channel in final design is 40 ft. wide with rip rap protection 25 feet downstream from the end sill of the stilling basin. After high flood discharges some maintenance of the channel bed can be expected.

Overall, the final design is considered hydraulically satisfactory and recommended for construction.

REFERENCES

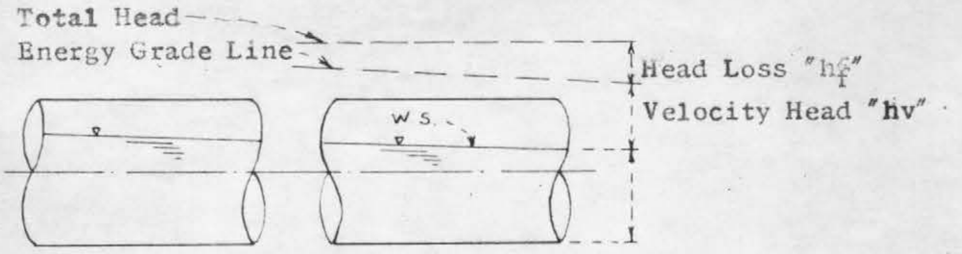
(1) Buckley, G. L., Hydraulic Model Studies of the Morning-Glory Spillway for Hungry Horse Dam. Hydraulic Lab. Report No. Hyd-355, U. S. Bureau of Reclamation, Denver, Colo. April 23, 1954.

(2) Tennessee Valley Authority. The Upper Holston Projects. Technical Report No. 14.

(3) Bureau of Reclamation. Hydraulic Model Studies of the Spillway and Outlet Works of Anchor Dam, Owl Creek Unit. Missouri River Basin. Hydraulic Lab. Report No. 289. Denver, Colo. February 1951.

TABLE 1

PROTOTYPE SPILLWAY BACKWATER COMPUTATIONS



DATA

1 S_o = 0.002
2 N = 0.012
3 Q = 11,000 7,000 3,000 c.f.s.
4 V (Initial) = 120 120 120. f.p.s.
5 Head Loss at the Beginning = 25 25 25 ft.

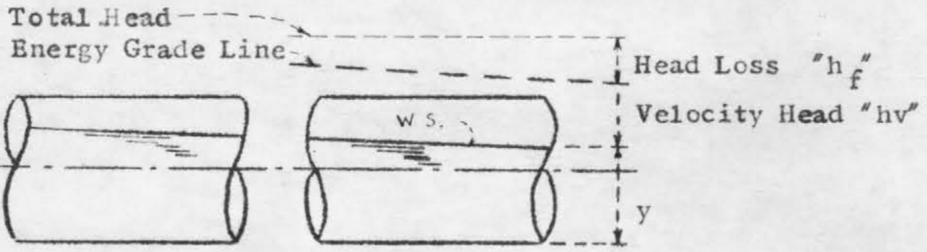
FORMULAS

$$h_v = \frac{V^2}{2g} \qquad S_f = \left(\frac{n}{1.49}\right)^2 \frac{V^2}{R^{4/3}} \qquad \Delta X = \frac{(y_2 + h_{v2}) - (y_1 + h_{v1})}{S_o - S_f} \qquad h_f = S_f \Delta X$$

1 Q	2 y_1	3 h_{v1}	4 y_2	5 h_{v2}	6 V_2	7 Avg. S_f	8 $S_o - S_f$	9 E.G.L. 4+5	10 E.G.L. 2+3	11 9 − 10	12 ΔX 11/8	13 $\Sigma \Delta X$	14 h_f 7×12	15 Σh_f Tunnel	16 E.G.L. + Losses 9 + 14
11,000	7.60	223	7.65	220	118.8	0.159	-.157	227.7	230.6	- 2.9	18.5	18.5	2.94	2.94	230.64
	7.65	220	7.75	213	117.0	.155	-.153	220.8	227.7	- 6.9	45.2	63.7	7.00	9.94	227.80
	7.75	213	7.85	207	115.2	.147	-.145	214.9	220.8	- 5.9	40.7	104.4	5.98	15.92	220.88
	7.85	207	8.00	197	112.6	.140	-.138	205.0	214.9	- 9.9	71.9	176.3	10.04	25.96	215.04
	8.00	197	8.15	188	110.0	.133	-.131	196.2	205.0	- 9.2	69.2	245.5	9.23	35.19	205.43
	8.15	188	8.40	176	106.3	.122	-.120	184.4	196.2	-11.8	98.4	343.9	12.00	47.19	196.40
	8.40	176	8.70	163	102.3	.111	-.109	171.7	184.4	-12.7	116.5	460.4	12.90	60.09	184.60
	8.70	163	9.00	150	98.0	.100	-.098	159.0	171.7	-12.7	127.0	587.4	12.70	72.79	171.70
	9.00	150	9.50	134	92.7	.088	-.086	143.5	159.0	-15.5	176.3	763.7	15.50	88.29	159.00
	9.50	134	10.00	119	87.4	.077	-.075	129.0	143.5	-14.5	193.0	956.7	14.85	103.14	143.85
	10.00	119	10.05	118	87.0	.070	-.068	128.1	129.0	- 0.9	12.9	969.6	0.91	104.05	129.01
7,000	5.38	223	5.50	219	118.5	0.208	-.206	224.5	228.4	- 3.9	18.9	18.9	3.93	3.93	228.43
	5.50	219	5.70	199	113.0	.189	-.187	204.7	224.5	-19.8	106.0	124.9	20.00	23.93	224.70
	5.70	199	6.00	175	106.1	.166	-.164	181.0	204.7	-23.7	144.5	269.4	24.00	47.93	205.00
	6.00	175	6.20	160	101.4	.144	-.142	166.2	181.0	-14.8	104.3	373.7	15.00	62.93	181.20
	6.20	160	6.50	141	95.0	.125	-.123	147.5	166.2	-18.7	152.0	525.7	19.00	81.93	166.50
	6.50	141	6.80	126	90.0	.107	-.105	132.8	147.5	-14.7	140.0	665.7	15.00	96.93	147.80
	6.80	126	7.00	117	86.6	.094	-.092	124.0	132.8	- 8.8	95.7	761.4	[illegible].00	105.93	133.00
	7.00	117	7.30	105	82.0	.083	-.081	112.3	124.0	-11.7	144.6	906.0	12.00	117.93	124.30
	7.30	105	7.40	102	80.8	.075	-.073	109.4	112.3	- 2.9	39.7	945.7	2.98	120.91	112.38
	7.40	102	7.45	100	80.2	.073	-.071	107.5	109.4	- 1.9	26.8	972.5	1.97	122.88	109.47
3,000	3.00	223	3.10	200	113.4	0.422	-.420	203.1	226.0	-22.9	54.6	54.6	23.00	23.00	226.10
	3.10	200	3.20	185	109.0	.330	.328	188.2	203.1	-14.9	45.5	100.1	15.00	38.00	203.20
	3.20	185	3.30	165	103.0	.290	.288	168.3	188.2	-19.9	69.0	169.1	20.00	58.00	188.30
	3.30	165	3.40	154	99.5	.250	.248	157.4	168.3	-10.9	44.0	213.1	11.00	69.00	168.40
	3.40	154	3.50	140	94.9	.230	.228	143.5	157.4	-13.9	61.0	274.1	14.00	83.00	157.50
	3.50	140	3.60	130	91.3	.197	.195	133.6	143.5	- 9.9	50.3	324.4	9.93	92.93	143.53
	3.60	130	3.80	112	85.0	.168	.166	115.8	133.6	-17.8	107.0	431.4	17.97	110.90	133.77
	3.80	112	4.00	100	80.2	.140	.138	104.0	115.8	-11.8	85.4	516.8	11.96	122.86	115.96
	4.00	100	4.25	84	73.4	.118	.116	88.3	104.0	-15.7	135.0	651.8	15.94	138.80	104.24
	4.25	84	4.50	71	67.5	.093	.091	75.5	88.3	-12.8	141.0	792.8	13.10	151.90	88.60
	4.50	71	4.70	64	64.1	.075	.073	68.7	75.5	- 6.8	93.2	886.0	6.97	158.87	75.67
	4.70	64	4.85	58	61.0	.068	.066	62.85	68.7	- 5.8	87.6	973.6	5.96	164.83	68.81

TABLE 2

MODEL SPILLWAY BACKWATER COMPUTATIONS



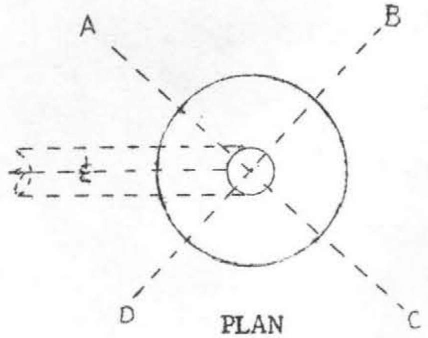
DATA

1	S_o = 0.002				
2	N = 0.008				
3	Q (Model)	= 2.007	1.278	0.547	cfs
4	V_m (Initial)	= 21.4	21.4	21.4	fps
5	Head Loss at the beginning	= 0.8	0.8	0.8	ft.

FORMULAS

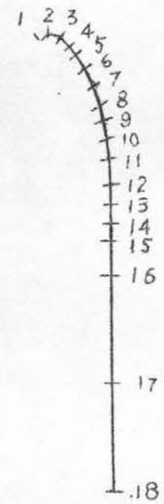
$$h_v = \frac{V^2}{2g} \qquad S_f = \left(\frac{n}{1.49}\right)^2 \frac{V^2}{R^{4/3}} \qquad \Delta X = \frac{(y_2 + h_{v2}) - (y_1 + h_{v1})}{S_o - S_f} \qquad h_f = S_f \Delta X$$

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Q	y_1	h_{v1}	y_2	h_{v2}	V_2	Avg. S_f	$S_o - S_f$	E.G.L. 4+5	E.G.L. 2+3	9 − 10	ΔX 11/8	$\Sigma \Delta X$	h_f 7 × 12	Σh_f Tunnel	E.G.L. + Losses 9 + 14
2.007 (11,000)	0.249	7.150	0.255	6.700	20.8	0.210	0.208	6.955	7.399	−0.444	2.13	2.13	0.447	0.447	7.402
	.255	6.700	.270	5.850	19.4	.188	.186	6.120	6.955	−0.835	4.49	6.62	0.845	1.292	6.965
	.270	5.850	.300	4.500	17.0	.145	.143	4.800	6.120	−1.320	9.22	15.84	1.335	2.627	6.135
	.300	4.500	.323	3.800	15.6	.107	.105	4.123	4.800	−0.677	6.45	22.29	0.690	3.317	4.813
1.278 (7,000)	0.175	7.150	0.185	6.200	19.9	0.275	0.273	6.385	7.325	−0.940	3.44	3.44	0.946	0.946	7.331
	.185	6.200	.200	5.050	18.0	.217	.215	5.250	6.385	−1.135	5.28	8.72	1.148	2.094	6.398
	.200	5.050	.220	3.900	15.8	.163	.161	4.120	5.250	−1.130	7.02	15.74	1.143	3.237	5.263
	.220	3.900	.230	3.450	14.9	.127	.125	3.680	4.120	−0.440	3.52	19.26	0.447	3.684	4.127
	.230	3.450	.240	3.100	14.1	.108	.106	3.340	3.680	−0.340	3.21	22.47	0.347	4.031	3.687
0.547 (3,000)	0.096	7.150	0.110	5.000	17.9	0.481	0.479	5.110	7.246	−2.136	4.46	4.46	2.143	2.143	7.253
	.110	5.000	.120	3.900	15.8	.310	.308	4.020	5.110	−1.090	3.54	8.00	1.099	3.242	5.119
	.120	3.900	.135	2.770	13.3	.212	.210	2.905	4.020	−1.115	5.31	13.31	1.126	4.368	4.031
	.135	2.770	.150	2.050	11.5	.136	.134	2.200	2.905	−0.705	5.26	18.57	0.705	5.073	2.905
	.150	2.050	.160	1.700	10.4	.095	.093	1.860	2.200	−0.340	3.66	22.23	0.348	5.421	2.208



NOTE

For location of Piezometer see Fig. 68



PIEZOMETER LOCATION

TABLE 3
MODEL PRESSURES IN FEET OF WATER
PRELIMINARY CREST WITHOUT DEFLECTOR
CREST DIAMETER = 56′ SHAFT DIAMETER = 15′

Q cfs	PIEZ LINE	PIEZOMETER NUMBER 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
3,000	A	2.9	1.3	0.8	0.5	-0.3	0	-0.8	-	-0.3	-0.3	-1.0	-0.3	0.3	-0.3	0	0	-0.3	-0.3
	B	3.1	1.3	0.5	0.5	0	0.5	-0.5	-1.0	-0.8	-0.8	0.5	-0.8	0.3	-0.8	0	0	-0.8	-0.5
	C	2.6	1.0	0.8	0.5	-0.3	0.5	-0.5	-1.0	-0.8	-1.5	0.8	-0.5	1.8	-0.3	-0.3	0.8	-0.3	0
	D	2.9	1.6	0.8	0.5	0.3	0.3	0	-0.8	-0.3	-0.3	0.5	-0.3	0.3	-0.3	0	0.8	-0.3	0.3
7,000	A	3.4	0.8	0.8	0.5	-0.5	0.3	1.0	-	-1.3	-1.0	-0.8	0.5	2.1	0.3	0.8	-0.8	-0.3	-0.5
	B	4.2	0.8	0.5	0.5	-0.3	0.8	-0.8	-2.1	-2.1	-1.3	3.1	4.2	3.9	0.8	2.1	0.5	-1.8	-0.8
	C	3.1	1.0	0.8	0.5	-0.3	0.8	-0.5	-1.0	-1.3	-1.8	1.3	-0.5	3.1	0.8	1.0	2.1	0	-1.0
	D	1.0	1.6	1.0	0.8	0.3	1.0	0.8	0	0.8	1.0	1.8	0.5	1.0	0	0.5	3.1	0	0.3
9,000	A	3.1	0.3	0.3	0.3	-0.8	0.3	-1.0	-	-1.3	0.8	2.9	2.6	3.1	1.6	1.3	1.8	-3.9	-4.2
	B	4.2	0.3	0	0.3	0.5	0.8	-1.0	-2.3	-0.3	-0.3	7.3	4.7	1.6	-1.8	-2.1	-4.7	-3.1	-4.2
	C	2.9	0.5	0.3	0.3	-0.5	-0.8	-0.8	-1.8	-1.3	0.3	5.2	3.7	4.7	0.3	-1.0	-0.3	-3.1	-3.4
	D	3.9	1.6	1.0	1.0	0.8	1.6	1.3	0.5	1.3	1.6	3.4	4.4	5.2	0.8	-1.3	-0.5	-3.9	-4.2
10,000	A	3.1	-0.3	0	0	-1.0	0.3	-1.0	-	1.3	5.7	5.5	1.8	-2.1	-6.5	-7.6	-11.7	-12.0	-12.0
	B	4.2	0	0	0.3	-0.8	0.8	-1.0	-1.0	4.2	6.8	6.3	-1.0	-5.5	-10.4	-10.4	-14.4	-14.6	-11.2
	C	2.9	0.5	0.3	0.3	-0.8	0.8	-0.5	-1.0	1.3	3.9	7.0	1.8	-0.5	-7.0	-9.4	-10.4	-12.0	-11.2
	D	3.9	1.6	1.0	1.0	1.0	1.8	1.8	1.8	3.1	5.2	7.0	3.1	-1.6	-7.0	-9.9	-9.6	-11.2	-11.0
11,000	A	2.9	-0.5	-0.3	0	-1.0	0.3	-0.8	-	3.4	3.4	0.5	-7.0	-13.8	-21.4	-23.0	-26.1	-25.6	-21.9
	B	4.2	-0.5	-0.3	0	-0.8	0.8	-0.5	1.3	5.7	3.4	1.0	-6.8	-16.2	-22.4	-24.8	-29.5	-28.0	-21.7
	C	2.9	0.3	0	0	-0.8	0.8	0	0	2.1	1.6	2.6	-6.3	-11.0	-19.6	-24.8	-24.5	-26.1	-21.7
	D	3.9	1.3	1.0	-1.0	0.8	2.3	2.6	2.3	3.9	3.7	2.3	-5.0	-	-20.1	-22.7	-24.8	-26.1	-21.4

- PIEZOMETER DID NOT FUNCTION PROPERLY

TABLE 4

TABLE OF CREST PRESSURES IN FEET OF WATER

PRELIMINARY CREST WITH VARIOUS DEFLECTOR SIZES

CREST DIAMETER = 56'   SHAFT DIAMETER = 15'

DEFL SIZE	Q cfs	LINE	PIEZOMETER NUMBER 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
$2\frac{1}{2}$"	3000	A	2.9	1.3	0.8	0.5	-0.3	0	-0.8	---	-0.5	-0.5	-1.3	0	0.3	-0.3	-0.3	0	-0.3	0
		B	3.1	1.0	0.5	0.5	-0.3	3.1	-0.5	-1.0	-0.8	-0.8	0.5	-0.8	0	-0.5	-0.3	-0.3	-1.0	-0.3
		C	2.6	1.0	0.8	0.3	-0.3	0.3	-0.5	-1.0	-0.8	-1.8	0.5	-0.5	1.6	-0.5	-0.3	0.8	-0.3	0.3
		D	2.9	1.3	0.8	0.5	0.3	0.3	0	-0.8	0	0	0.5	0	0.3	-0.3	-0.3	0.8	-0.3	0
	5000	A	3.1	1.0	0.8	0.5	-0.3	0.3	-1.0	---	-1.0	-0.5	-1.0	-0.3	0.3	0	0.3	0.8	0.5	0.3
		B	3.7	1.0	0.5	0.8	-0.3	0.8	-0.5	-1.6	-1.6	-1.3	0.5	-0.8	1.6	1.6	1.8	0.3	-0.5	0.5
		C	2.9	1.0	0.8	0.5	-0.3	0.3	-0.8	-1.3	-1.0	-1.8	0.8	-0.8	1.8	-0.5	-0.3	0.8	0.5	0.8
		D	3.1	1.6	0.8	0.8	0.3	0.8	0.3	-0.8	-0.3	0.3	1.0	0	0.5	0	0.8	0.8	0.5	1.0
	7000	A	3.1	0.8	0.5	0.5	-0.5	0.3	-1.0	---	-1.0	-0.5	-1.0	0.5	2.9	1.6	1.8	1.6	2.6	3.1
		B	3.9	0.8	0.3	0.5	-0.3	0.8	-0.8	-1.8	-2.1	-1.3	4.7	5.0	4.2	1.8	3.1	1.6	0	1.8
		C	3.1	0.8	0.5	0.5	-0.5	0.3	-0.8	*	-1.6	-1.8	1.3	0	3.9	2.1	1.8	3.9	1.8	2.6
		D	3.7	1.6	1.0	1.0	0.3	1.0	0.8	0	0.8	0.8	1.8	0.8	1.6	0.8	1.6	4.2	2.6	3.4
	9000	A	3.1	0.3	0.3	0.3	-0.8	0.3	-1.3	---	0.3	5.7	10.7	8.6	6.3	3.9	4.2	4.4	39.7	68.4
		B	4.2	0.3	0	0.5	-0.5	0.8	-0.8	-1.8	2.3	8.4	11.5	7.6	5.0	2.1	2.6	4.4	38.4	65.0
		C	3.1	0.8	0.5	0.3	-0.5	0.8	-0.5	1.3	1.3	5.5	10.7	8.9	8.9	4.4	3.7	7.6	38.2	65.8
		D	3.7	1.6	1.0	1.6	0.5	1.8	1.6	1.3	3.1	7.6	11.7	9.6	7.6	5.0	4.2	7.0	38.2	69.7
$1\frac{7}{8}$"	3000	A	2.9	1.0	0.8	0.5	-0.3	0	-0.8	---	-0.5	-6.5	-1.3	-0.3	0.3	-0.3	-0.3	0	0	0
		B	3.1	1.0	0.5	0.5	-0.3	0.5	-0.5	-1.0	-0.8	-0.8	0.5	-0.8	0.3	-0.8	-0.3	0	-0.5	-0.3
		C	2.6	1.0	0.8	0.5	-0.3	0.3	-0.5	-1.0	-0.8	-1.6	0.8	-0.5	1.6	-0.5	-0.3	0.5	-0.3	0
		D	2.9	1.3	0.8	0.5	0.3	0.3	0	-0.8	0	0	0.5	0	0.3	-0.3	-0.3	0.5	-0.5	0
	5000	A	3.1	1.0	-0.8	0.5	-0.3	0.3	-1.0	---	-1.0	-0.8	-1.3	-0.5	0.3	0.3	0.8	0.5	0.5	0
		B	3.7	1.0	0.5	0.8	-0.3	0.8	-0.5	-1.6	-1.3	1.3	-0.5	-1.6	2.1	2.1	1.8	0.3	-0.8	0
		C	2.9	1.0	0.8	0.5	-0.3	0.5	-0.5	-1.6	1.0	-1.8	0.8	-0.8	1.8	-0.5	-0.3	0.8	0.5	0.5
		D	3.4	1.6	1.0	0.8	0.3	0.5	0.3	-0.8	0.3	0.3	1.0	0	0.8	0	0.8	0.8	0.3	1.0
	7000	A	3.4	0.8	0.5	0.5	-0.5	0.3	-1.0	---	-1.0	-0.8	-0.8	1.3	2.3	1.3	1.6	1.6	2.3	1.3
		B	3.9	0.8	0.5	0.8	-0.3	0.8	-0.8	-1.8	-2.1	-1.3	3.9	4.7	3.9	2.3	2.9	0.5	-0.8	0
		C	3.1	1.0	0.8	0.5	-0.3	0.8	-0.5	-1.6	-1.3	-1.8	1.6	0	4.4	2.6	-0.5	3.1	1.0	0.8
		D	3.7	1.6	1.0	0.8	0.3	1.0	0.8	0	0.8	1.0	1.8	0.5	1.3	0.8	1.3	4.2	1.8	2.3
	9000	A	3.4	0.5	0.5	0.3	-0.5	-0.3	-1.3	---	-1.3	1.3	5.0	6.3	7.6	3.9	2.9	-1.3	-1.0	-0.5
		B	4.2	0.3	0.3	0.5	-0.5	0.8	-0.8	-2.1	-0.3	6.0	5.5	6.8	4.2	0	0.3	-1.0	-2.1	0
		C	3.4	0.8	0.5	0.5	-0.3	0.8	-0.5	-1.6	-1.0	0	5.5	6.0	7.3	3.4	2.6	2.6	0	1.3
		D	3.9	1.6	1.0	1.0	0.5	1.6	1.6	1.0	1.8	2.3	5.2	8.9	7.3	3.7	2.3	2.1	-0.5	2.1
	11,000	A	4.7	1.6	3.7	5.7	7.3	11.5	13.8	---	12.8	9.9	6.8	2.3	-0.3	-3.1	-2.3	5.7	35.2	63.7
		B	5.5	1.6	3.7	5.7	7.3	11.7	13.6	13.6	12.3	9.9	8.9	2.6	-0.3	-4.7	-2.9	5.2	32.9	61.3
		C	4.7	2.9	5.2	6.8	8.6	12.8	14.9	14.9	13.3	9.9	9.6	2.6	1.6	-1.0	-4.2	5.0	35.0	62.9
		D	6.5	5.2	6.3	7.8	9.4	12.8	15.4	15.1	13.6	10.7	8.9	3.4	0.3	-1.6	-2.1	5.0	35.0	63.9
	11,500	A	13.0	11.5	13.0	14.1	15.7	18.8	20.9	---	19.1	15.9	2.3	3.1	4.7	1.3	2.1	8.9	37.4	65.8
		B	13.8	11.2	12.8	14.1	15.4	19.1	20.9	20.6	18.5	15.7	14.6	7.6	4.4	-1.0	2.1	9.9	37.9	66.8
		C	15.7	12.3	13.6	14.9	16.2	19.6	21.4	21.2	19.1	15.4	14.9	7.6	6.0	0.8	0.3	12.3	39.2	94.0
		D	14.4	13.6	14.1	15.1	17.0	19.6	21.7	21.2	19.6	16.2	14.4	3.1	4.4	1.0	1.0	8.6	37.4	95.6
$1\frac{1}{2}$"	7000	A	3.4	0.8	0.5	0.5	-0.5	0.3	-1.0	---	-1.3	-1.0	-1.0	0.8	2.3	1.0	1.5	1.3	0.8	0.5
		B	3.7	0.8	0.5	0.5	-0.3	0.8	-0.8	-1.8	-2.1	-1.3	3.7	4.4	4.2	2.1	1.8	0.5	0.5	-0.3
		C	3.1	1.0	0.8	0.5	-0.3	0.8	-0.5	-1.8	-1.3	-1.8	1.3	-0.3	3.1	1.8	1.6	3.1	0.8	0
		D	3.7	1.6	1.0	1.0	0.3	1.0	0.8	-0.8	0.8	0.8	1.8	0.5	1.6	0.5	0.8	3.7	1.0	1.6
	9000	A	3.4	0.5	0.3	0.5	-0.5	0.3	-1.3	---	-1.0	0.8	2.9	3.9	5.0	2.6	2.1	-1.3	-2.9	-3.4
		B	3.9	0.3	0.3	0.5	-0.5	0.8	-1.0	-2.1	0	5.2	8.4	6.0	2.9	-0.5	-0.5	-2.6	-4.2	-1.8
		C	3.4	0.8	0.5	0.5	-0.5	0.8	-0.5	-1.6	-1.0	0.5	5.7	5.0	5.7	1.6	0	0.8	-2.1	-1.8
		D	4.2	1.6	1.0	1.0	0.5	1.6	1.6	0.8	1.8	2.1	3.9	7.3	6.0	1.8	0.8	-0.3	-4.4	-2.6
	11,000	A	3.1	-0.5	0	0	-0.8	0.5	-0.3	---	6.0	4.2	0.3	-5.2	-9.4	-13.6	-14.9	-18.0	-9.6	20.1
		B	4.2	-0.5	-0.3	0.3	-0.8	1.0	-0.8	5.0	7.6	4.2	2.3	-5.7	-10.7	-15.9	-16.2	-18.3	-11.0	18.3
		C	3.1	0.5	0.3	0.5	-0.5	1.3	1.0	3.1	5.5	4.2	5.0	-2.9	-5.0	-12.8	-14.6	-13.8	-6.8	22.7
		D	4.2	1.3	1.3	1.3	1.0	2.9	3.7	4.4	6.0	6.8	6.0	-1.0	-6.0	-11.5	-14.1	-14.6	-9.9	17.8
1"	9000	A	4.2	*	*	*	-0.5	*	*	*	-1.3	1.0	3.7	3.7	5.0	2.1	2.1	0.5	-2.6	-2.9
		B	4.4	0.5	0.3	0.5	-0.3	1.0	-0.8	-2.1	-2.3	4.7	8.1	7.3	1.3	-1.0	-1.8	-3.1	-3.9	-2.9
		C	3.4	1.0	0.8	0.5	-0.3	1.0	-0.5	-1.3	-1.0	0.8	6.3	5.0	5.5	1.0	0	2.6	-1.6	-0.8
		D	3.7	*	*	*	0.8	*	*	*	*	2.1	5.0	4.7	6.0	1.8	1.3	0.5	-2.3	-1.8
	10,000	A	3.4	*	*	*	*	*	-0.5	*	*	*	3.9	*	0	-5.7	-7.3	-9.1	-11.0	-8.6
		B	4.4	0.3	0	0.5	0.5	1.0	-0.8	-0.8	-3.1	7.0	6.8	3.7	-3.9	-9.1	-8.6	-12.0	-13.0	-8.1
		C	3.4	1.0	0.5	0.5	-0.3	1.0	0	-0.8	1.6	4.4	7.0	3.4	0.5	-6.5	-9.1	-8.4	-9.6	-6.0
		D	3.7	*	*	*	*	*	*	*	*	*	*	-0.8	-1.0	-5.2	-7.6	-7.8	-9.4	-8.4
	11,000	A	3.4	*	*	*	*	*	*	*	5.5	*	*	-5.2	*	*	15.4	-20.1	-17.5	-7.6
		B	4.2	-0.3	0	0.3	-0.5	1.0	0	2.3	6.3	3.7	1.6	-1.8	-13.8	-18.5	-19.6	-23.5	-20.1	-8.6
		C	3.1	0.8	0.5	0.5	0.5	1.3	0.5	1.8	4.2	3.4	4.4	-3.7	-7.8	-15.9	-18.8	-17.0	-16.4	-6.0
		D	3.4	*	*	*	*	*	*	*	5.5	*	*	7.3	*	*	-17.5	-17.8	-17.8	-7.3
$1\frac{3}{4}$"	7000	A	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
		B	4.2	1.0	0.5	0.8	-0.3	0.8	-0.8	-1.8	-1.3	-1.0	4.7	1.0	3.4	1.8	2.1	0.8	-0.5	0.5
		C	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
		D	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*	*
	9000	A	3.4	0.8	0.3	0.3	-0.8	0.5	-0.8	---	-1.3	1.0	4.2	4.4	6.8	3.9	3.4	-0.5	-0.8	1.3
		B	3.7	0.3	0	0.5	-0.5	0.8	-1.0	-2.3	-0.3	4.7	8.1	7.3	2.6	-0.8	-0.3	-2.1	-1.8	1.8
		C	3.1	0.8	0.5	0.5	0.5	0.5	-0.5	-1.6	-1.0	0.3	6.0	6.0	7.0	2.9	1.3	2.1	1.3	2.1
		D	3.9	1.6	1.0	1.0	0.5	1.6	1.6	0.8	1.6	1.6	3.9	5.0	7.3	2.3	1.3	2.1	0.3	1.8
	11,000	A	3.4	0.3	0.8	1.8	3.1	8.1	10.4	---	9.1	6.3	3.1	0	-2.3	-5.7	-6.0	-6.5	23.0	54.0
		B	4.7	0.3	1.0	2.6	1.6	9.1	10.7	10.2	8.3	5.5	5.0	0.8	-5.2	-8.9	-8.9	-6.8	21.4	53.5
		C	3.1	0.8	1.3	2.3	3.4	8.9	12.0	13.0	11.7	8.4	8.4	0.8	-1.3	-7.6	-8.9	-5.2	23.8	53.0
		D	4.4	2.1	2.3	3.7	5.0	9.6	12.4	12.8	11.5	8.6	6.8	-2.1	-2.6	-6.3	-7.3	-5.2	24.0	53.5
	15,000	A	*	*	*	*	*	*	*	*	*	*	*	1.6	-1.0	-5.7	-5.0	4.2	32.4	60.8
		B	*	*	*	*	*	*	*	*	*	*	*	2.6	-2.1	-5.7	-5.0	3.4	31.3	62.1
		C	6.5	5.2	7.0	8.6	9.9	13.8	15.7	15.4	13.3	9.4	9.1	1.6	-0.3	-5.7	-6.5	5.7	32.4	60.8
		D	*	*	*	*	*	*	*	*	*	*	*	1.0	-1.3	-5.7	-6.0	5.2	32.4	61.9

* DATA NOT TAKEN

TABLE 5

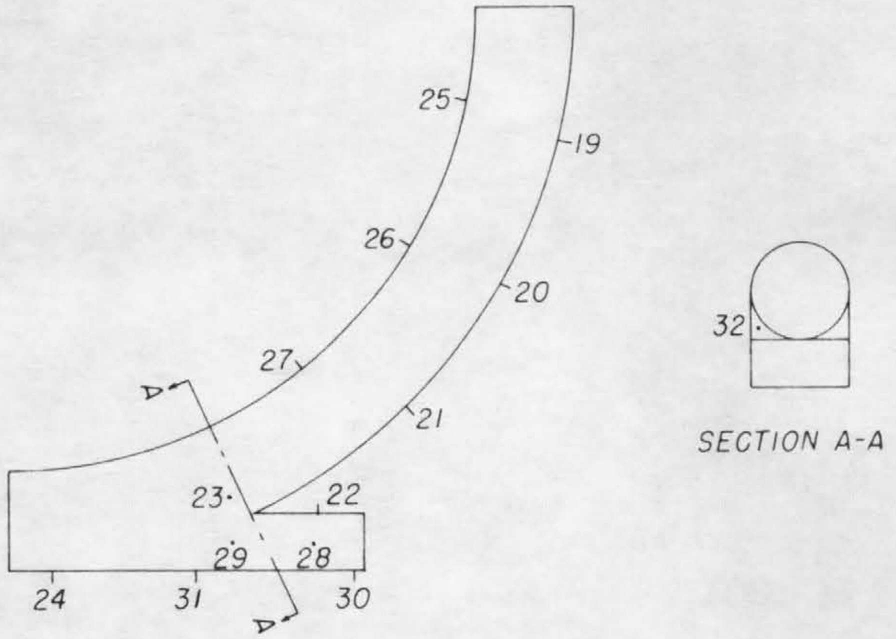
MODEL PRESSURES IN FEET OF WATER

RECOMMENDED CREST WITH RECOMMENDED DEFLECTOR (4.5')

CREST DIAMETER = 56' SHAFT DIAMETER = 15'

Q	PIEZ	PIEZOMETER NUMBER																	
cfs	LINE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
3200	A	2.9	0.8	0.5	0.3	-0.3	0.5	-0.3	*	-1.0	-0.8	-1.8	0	0.3	-0.5	-0.5	0.3	0.8	1.0
	B	3.4	1.3	0.5	0.5	0	0.5	-0.5	-1.3	-0.3	-1.3	0.5	0.5	0.8	-0.5	0	0.3	-0.3	0.5
	C	2.6	0.5	0.5	0.5	-0.3	0.3	-0.8	-1.0	-1.3	-1.8	-1.3	-0.5	0.3	-0.3	-0.8	0.8	0.8	1.8
	D	2.6	0.5	0	-0.3	-0.8	0	-1.0	-1.8	-1.6	-1.3	0	-1.3	0.5	-1.0	0	1.3	0.8	1.8
7000	A	3.1	0.3	0.3	0.3	-0.3	0.8	-0.5	*	-1.6	-1.3	-1.3	1.0	2.6	1.3	1.8	2.9	3.4	4.2
	B	3.7	0.5	0.3	0.3	-0.3	0.5	-0.8	-2.1	-1.3	-1.3	2.9	0.8	3.9	1.3	1.6	0.3	0.3	1.8
	C	2.6	0.5	0.3	0.3	-0.3	0.5	-0.8	-1.6	-1.3	-1.3	1.3	0.5	4.2	2.3	1.8	4.2	2.6	3.1
	D	2.6	0.5	0	-0.3	-0.8	0	-1.0	-1.8	-1.3	-1.0	0.3	1.8	1.8	0.5	1.6	3.9	2.3	3.1
9000	A	3.1	0	0	0	-0.5	0.8	-0.5	*	-1.6	0	3.1	5.7	4.7	-0.3	-2.3	-4.2	-1.6	3.1
	B	3.9	0	0	0.3	-0.5	0.5	-1.0	-2.3	0	4.4	8.4	6.5	2.3	-2.9	-2.9	-5.0	-2.3	1.6
	C	3.1	0.8	0	0.3	-0.5	0.5	-0.5	-1.6	-0.8	1.0	6.8	4.2	4.7	0	-0.8	-1.0	0	2.6
	D	3.4	0.8	0	0.5	-0.5	0.5	0	-0.5	0	2.9	5.7	4.2	5.0	1.0	-0.5	-0.3	-1.6	7.8
11000	A	3.1	0	0	0.8	1.0	5.5	7.8	*	7.3	4.7	0.8	-3.4	-6.8	-11.0	-9.9	-5.5	24.8	53.0
	B	3.9	-0.3	0.5	1.6	2.9	8.1	10.4	9.6	8.1	4.4	5.0	0.3	-1.0	-6.5	-6.5	-2.3	25.0	53.5
	C	3.1	0.5	0.3	0.8	0.8	5.7	9.4	10.7	9.4	5.7	6.0	-1.6	-3.9	-6.3	-9.9	-3.1	50.4	52.2
	D	4.2	1.8	1.3	1.8	7.6	8.1	11.7	13.0	11.5	8.6	6.5	-2.3	-3.7	-7.8	-8.1	-0.5	24.0	51.7
11500	A	3.9	1.6	3.1	5.0	6.3	11.0	12.5	13.6	10.2	7.3	3.4	-1.0	-4.4	-7.6	-7.0	0	29.5	58.5
	B	5.0	1.6	3.4	5.2	7.3	12.0	13.8	13.8	12.0	8.6	8.9	1.0	-0.3	-5.7	-5.0	1.6	30.0	58.5
	C	4.4	2.3	3.9	5.7	7.6	11.5	13.8	13.6	11.5	7.6	7.0	-0.3	-2.9	-8.4	-8.9	-0.3	29.0	59.0
	D	5.5	4.2	5.0	6.5	8.4	12.3	14.6	14.9	12.8	9.6	7.6	1.6	-3.1	-7.6	-7.3	0.5	30.8	59.0

* DATA NOT TAKEN



NOTES

For Piezometer Locations Refer To Fig. 75b

Piezometers 25, 26 and 27 Were Not Always In Contact With The Flow.

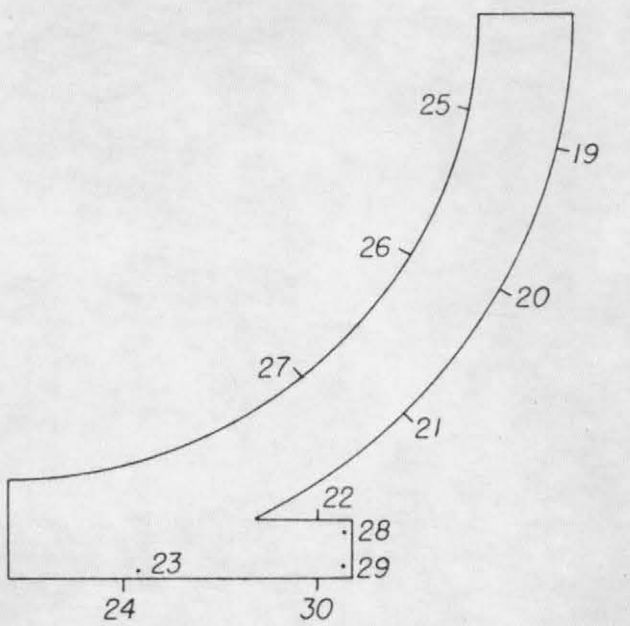
TABLE 6

MODEL PRESSURES AT THE SPILLWAY-OUTLET JUNCTION

OUTLET SIZE 8' x 15'

Ventilations at the gates and below the deflector only

Spillway	Outlet	Piezometer Readings													
Q	Q	19	20	21	22	23	24	25	26	27	28	29	30	31	32
0	1500	-	-	-	-	-	0	-	-	-	0.3	0.8	-2.3	-4.2	-
	2000	-	-	-	-	-	1.0	-	-	-	0	1.0	-2.6	-6.2	-
	3000	-	-	-	-	-	3.1	-	-	-	-1.6	-1.0	-5.2	-7.0	-
3000	0	5.0	15.4	15.7	0	0.5	7.6	-	-	-	2.3	2.3	6.0	40.5	0
	1000	5.7	16.2	15.1	-4.2	-0.5	7.3	-	-	-	-7.0	-1.0	-3.9	40.5	0
	2000	6.3	16.4	15.1	-3.4	-0.8	7.6	-	-	-	-2.9	-1.3	1.3	43.9	-1.0
	3000	12.3	16.7	15.4	-3.4	-1.6	-0.6	-	-	-	-6.8	-5.2	-4.4	-37.1	-4.2
5000	0	34.5	21.4	21.9	1.3	-0.3	6.9	-	-	-	5.0	3.4	9.1	95.3	2.6
7000	0	39.7	25.0	29.8	3.7	1.6	12.0	-	-	-	7.6	4.2	10.7	-	1.3
	1000	41.2	26.4	30.3	0.3	9.4	12.2	-	-	-	1.8	7.3	2.6	-	7.8
	2000	42.6	26.4	30.6	2.3	11.5	20.1	-	-	-	2.3	15.7	2.6	-	9.4
	3000	42.1	27.4	31.6	6.0	10.7	24.5	-	-	-	3.1	22.4	4.4	-	4.4
9000	0	13.4	12.1	13.7	2.6	2.0	6.5	-	-	-	4.2	2.7	5.4	41.2	3.2
	1000	34.5	32.6	37.1	24.3	8.9	25.3	-	-	-	-2.1	3.9	-6.5	-	8.4
	2000	13.7	13.2	15.0	12.4	12.1	24.2	-	-	-	12.9	15.4	13.3	23.2	12.9
	3000	36.3	41.5	32.9	94.3	91.7	83.5	-	4.2	29.2	93.5	104.4	94.8	108.8	87.7
11500	0	41.3	42.6	48.0	15.4	15.9	32.1	-	-	-	19.1	15.7	22.7	-	18.0
	1000	42.8	44.4	49.9	42.1	42.1	78.3	-	-	-	39.7	41.0	40.5	-	41.8



NOTES

For Piezometer Locations Refer to Fig. 75b
Piezometers 25, 26 and 27 Were Not Always In Contact With THe Flow.

TABLE 7

MODEL PRESSURES AT THE SPILLWAY-OUTLET JUNCTION

OUTLET SIZE 8' x 8'

Ventilation at the gates and below the deflector only

Spillway	Outlet	Piezometer Readings											
Q	Q	19	20	21	22	23	24	25	26	27	28	29	30
0.0	1000	-	-	-	-	-0.3	1.0	-	-	-	-	-6.3	2.1
	2000	-	-	-	10.7	1.6	1.6	-	-	-	-3.9	-4.7	17.5
	3000	-	-	-	8.1	1.6	3.4	-	-	-	-3.4	-6.0	10.4
3000	0	6.5	16.4	16.2	0.3	25.0	20.9	-	-	-	3.1	7.3	8.4
	1000	9.1	16.4	16.2	5.2	19.1	14.4	-	-	-	1.6	-3.9	13.3
	2000	23.8	16.4	16.2	14.6	13.0	11.2	-	-	-	-3.4	-2.6	22.2
	3000	24.3	16.4	16.2	25.0	12.0	11.5	-	-	-	-9.9	-4.7	16.7
5000	3000	30.8	22.4	23.0	46.2	18.5	17.8	-	-	-	46.5	50.9	57.2
7000	0	43.9	26.9	32.6	16.4	62.6	51.2	-	-	-	18.5	22.7	23.8
	1000	43.6	26.6	30.8	25.8	35.2	36.9	-	-	-	30.8	33.7	37.6
	2000	42.6	26.9	31.1	48.0	25.8	24.3	-	-	-	46.7	46.5	53.2
	3000	42.6	36.9	31.1	55.5	23.8	23.2	-	-	-	51.9	55.0	61.2
9000	0	32.6	32.9	37.4	23.8	74.4	62.1	-	-	-	25.0	30.0	31.1
	1000	33.2	33.2	37.6	42.3	40.2	40.0	-	-	-	43.6	46.5	49.3
	2000	33.4	31.8	37.9	55.1	33.4	31.3	-	-	-	57.2	57.4	60.8
	3000	34.7	35.0	39.7	68.9	31.8	31.6	-	-	-	63.4	68.4	74.9
11000	0	39.2	40.0	45.4	33.4	89.8	77.0	-	-	-	35.5	39.5	40.2
	1000	37.4	37.9	44.1	49.6	48.3	48.3	-	-	-	51.7	52.7	56.6
11500	2000	42.6	43.4	48.8	69.4	42.8	40.0	-	-	-	72.8	69.2	75.7
	2600	42.6	44.9	59.5	133.4	107.8	103.6	-	-	15.7	131.8	133.1	139.6

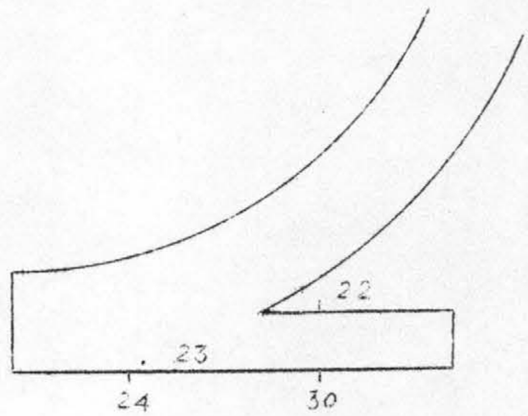


TABLE 8

MODEL PRESSURES AT THE SPILLWAY-OUTLET JUNCTION

Complete Ventilation of the Junction - Outlet Conduit 8'x 8'

Spillway Qs	Outlet Qo	22	23	24	30
0	1000	1.0	-0.8	0.5	8.1
	2000	1.3	0.5	1.8	8.6
	3000	28.0	2.9	2.1	11.5
3000	1000	---	32.6	35.2	4.2
9000	1000	17.2	40.0	31.6	20.9
	3000	46.7	33.2	33.2	54.0
11000	0	20.4	83.8	94.8	28.0
	1000	34.2	42.1	40.2	38.4
	2000	52.2	43.4	43.1	57.4
	2900	136.0	112.5	112.0	138.3

For pressures in vertical bend refer to Table 7

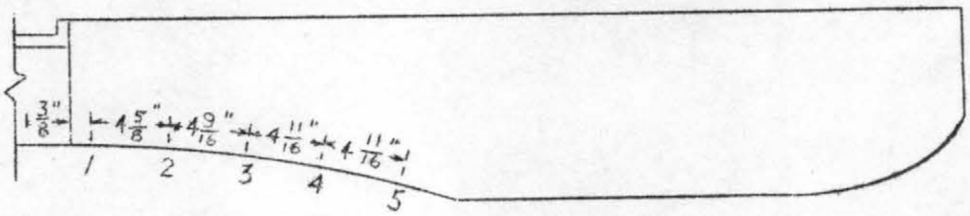


TABLE 9
MODEL PRESSURES IN FEET OF WATER ON TRANSITION FLOOR
FOR STILLING BASIN J

Transition Length = 60′
Basin Length = 80′
Basin Elevation = 8767′

(O) - Flow through outlet works

Q	Piezometer Number				
in c.f.s.	1	2	3	4	5
1,500 (O)	2.3	1.6	2.3	1.8	3.1
2,000 (O)	2.3	1.6	2.6	2.1	4.7
3,000 (O)	2.9	0.8	3.1	2.1	7.0
3,000	1.8	3.7	0	-0.3	8.1
5,000	0.8	4.4	-0.8	-1.0	13.8
7,000	0.5	4.4	-0.8	-1.3	18.5
9,000	0.3	4.7	-0.5	-0.3	22.7
10,000	0	4.7	-0.5	0	24.8
11,000	-0.3	4.7	1.0	-0.3	27.2
11,500	-0.8	4.7	-1.8	-0.8	28.0

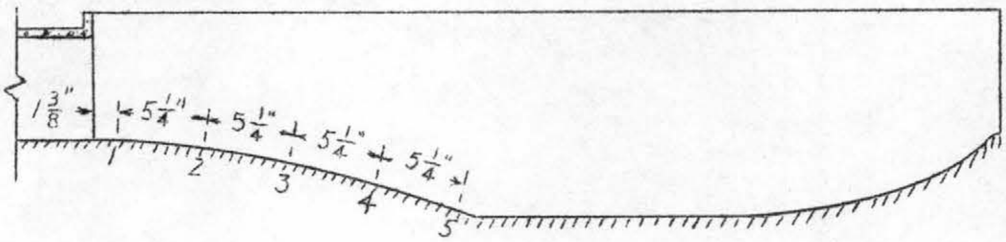


TABLE 10
MODEL PRESSURES IN FEET OF WATER ON TRANSITION FLOOR
FOR STILLING BASIN K

Transition Length = 60′
Basin Length = 80′

Basin Elevation = 8763.5′

(O) Flow through outlet works

Q in c.f.s.	PIEZOMETER NUMBER 1	2	3	4	5
1,000 (O)	5.0	5.2	7.5	11.3	16.4
2,000 (O)	7.4	1.6	3.6	10.0	16.3
3,000 (O)	9.8	0.8	-1.6	3.7	9.1
3,000	9.9	1.4	-1.6	1.0	3.1
5,000	13.0	-1.0	-1.5	1.7	5.5
7,000	14.9	-1.0	-1.6	0.8	1.1
9,000	16.7	-1.0	-1.6	0.5	1.1
11,000	18.8	-1.0	-1.7	0.5	1.4

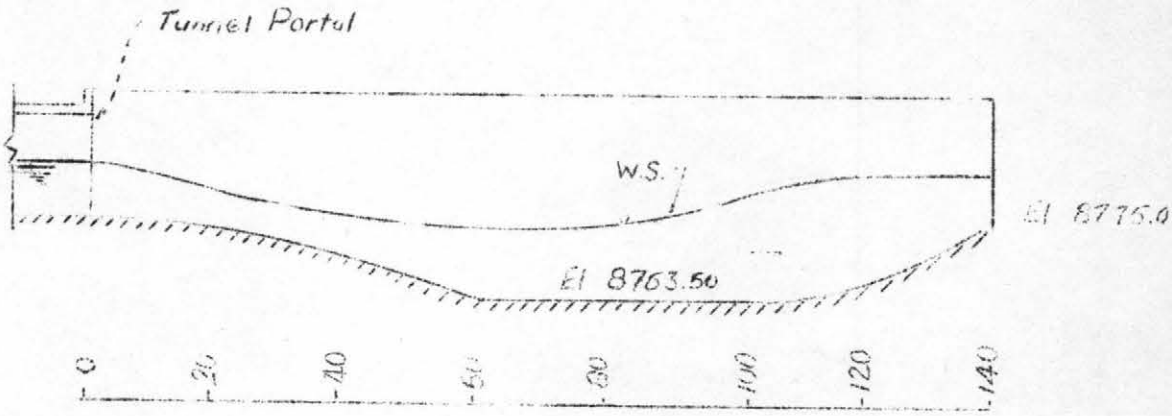
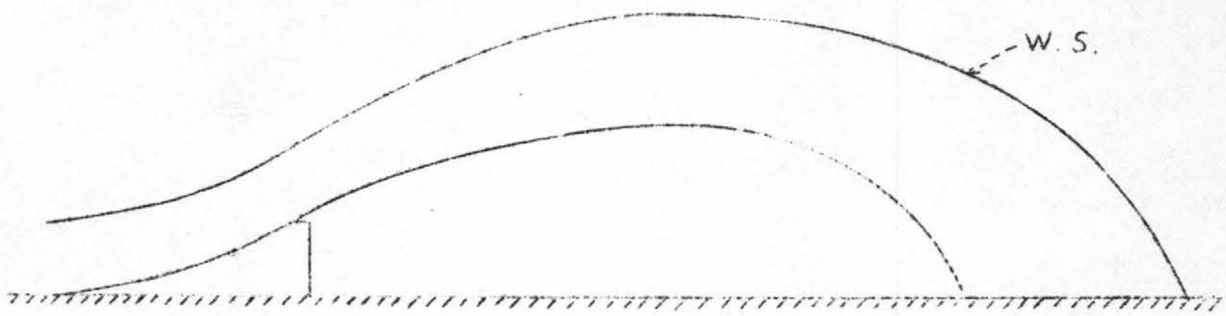


TABLE 11

WATER SURFACE PROFILES IN STILLING BASIN K

DISTANCE	DISCHARGE Q.			
	1000	2000	3000	14000
0	8779.5	8781.0	8784.0	8791.0
10	8779.5	8780.0	8781.0	8790.5
20	8778.5	8778.0	8780.0	8790.0
30	8779.5	8778.0	8779.0	8789.5
40	8780.5	8779.0	8777.0	8788.0
50	8780.0	8779.0	8775.0	8786.0
60	8780.0	8779.0	8775.0	8783.5
70	8780.0	8780.0	8774.0	8781.0
80	8780.0	8780.0	8775.0	8781.0
90	-	8781.0	8777.0	8785.0
100	-	8782.0	8780.0	8788.0
110	-	8782.0	8782.0	8791.0
120	-	8782.0	8784.0	8793.5
130	-	8783.0	8784.0	8794.5
140	-	8782.0	8783.0	8795.0



NOTES

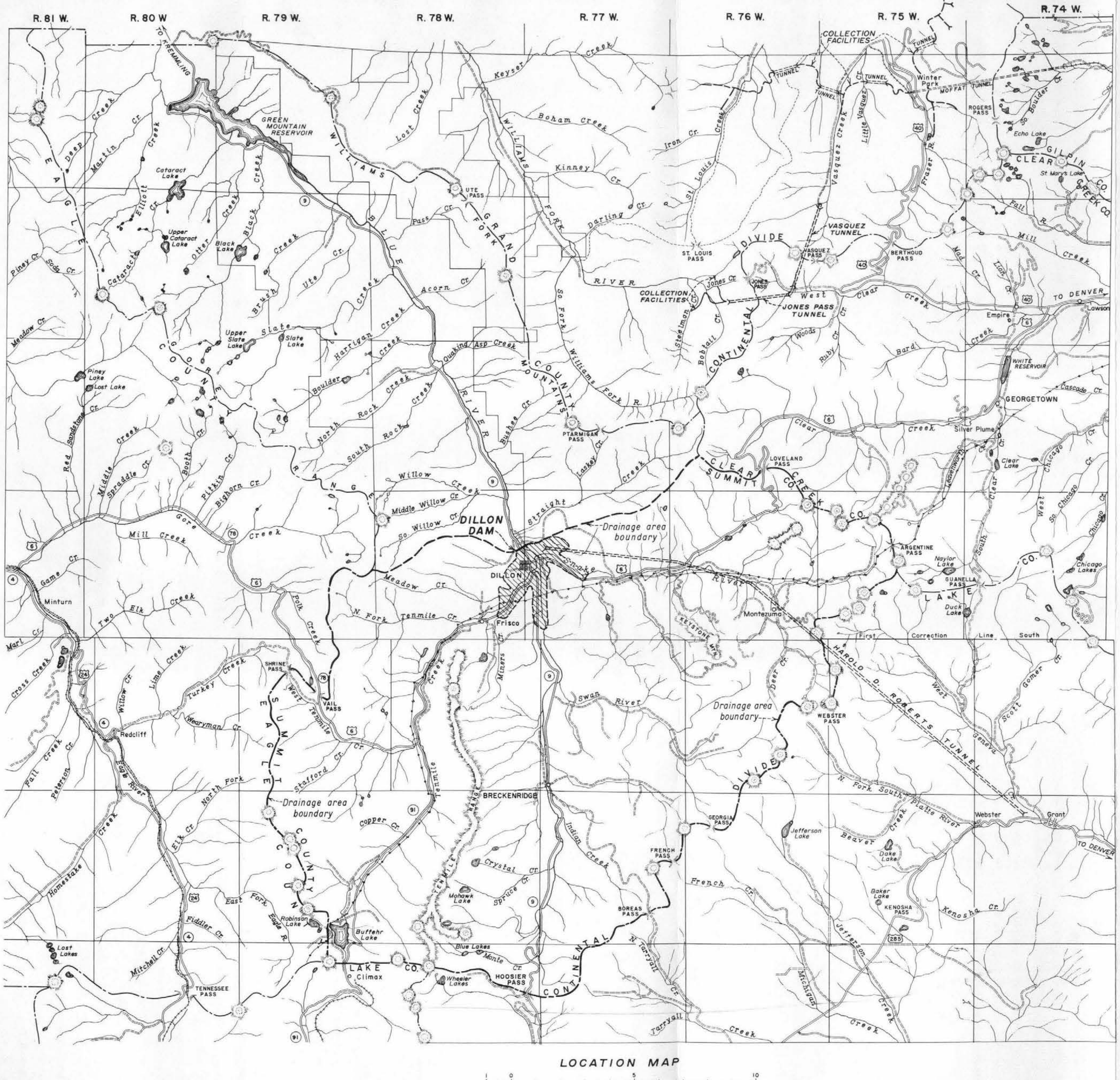
Distances are from tunnel portal
Water surface profiles are in elevations in ft.

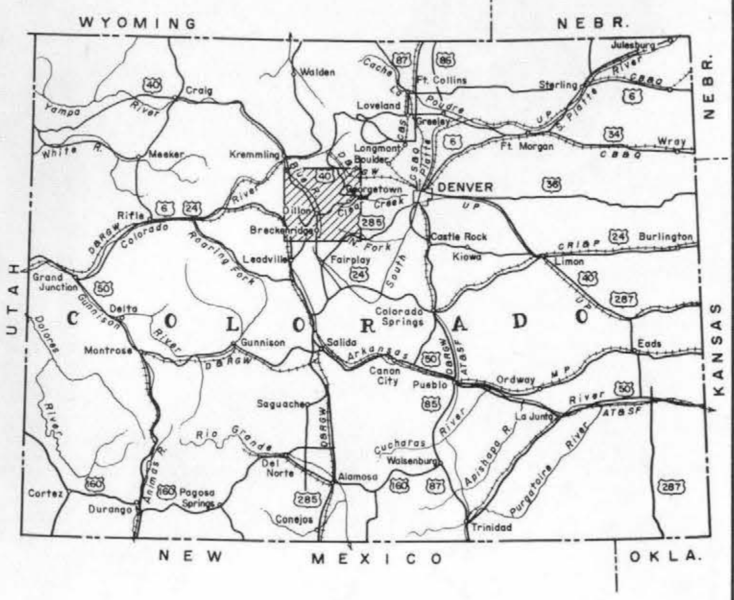
TABLE 12

WATER SURFACE PROFILES OF THE JET TRAJECTORY

STILLING BASIN K

DISTANCE	DISCHARGE IN C.F.S.				
	3500	5000	7000	9000	11000
140	8786	8795	8790	8785	8789
150	8789	8800	8798	8795	8795
160	8791	8800	8800	8800	8803
170	8791	8800	8803	8803	8808
180	8789	8800	8803	8805	8810
190	8786	8798	8802	8803	8810
200	8782	8795	8801	8800	8808
210	-	8790	8796	8798	8808
220	-	8785	8790	8794	8805
230	-	8780	8783	8791	8803
240	-	-	8782	8785	8802
250	-	-	8781	8783	8798
260	-	-	8781	8783	8796
270	-	-	-	8783	8790
280	-	-	-	-	8785





INDEX MAP

SCALE OF MILES

BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC. - ENGINEERS
DILLON DAM

LOCATION MAP

FIG. 1

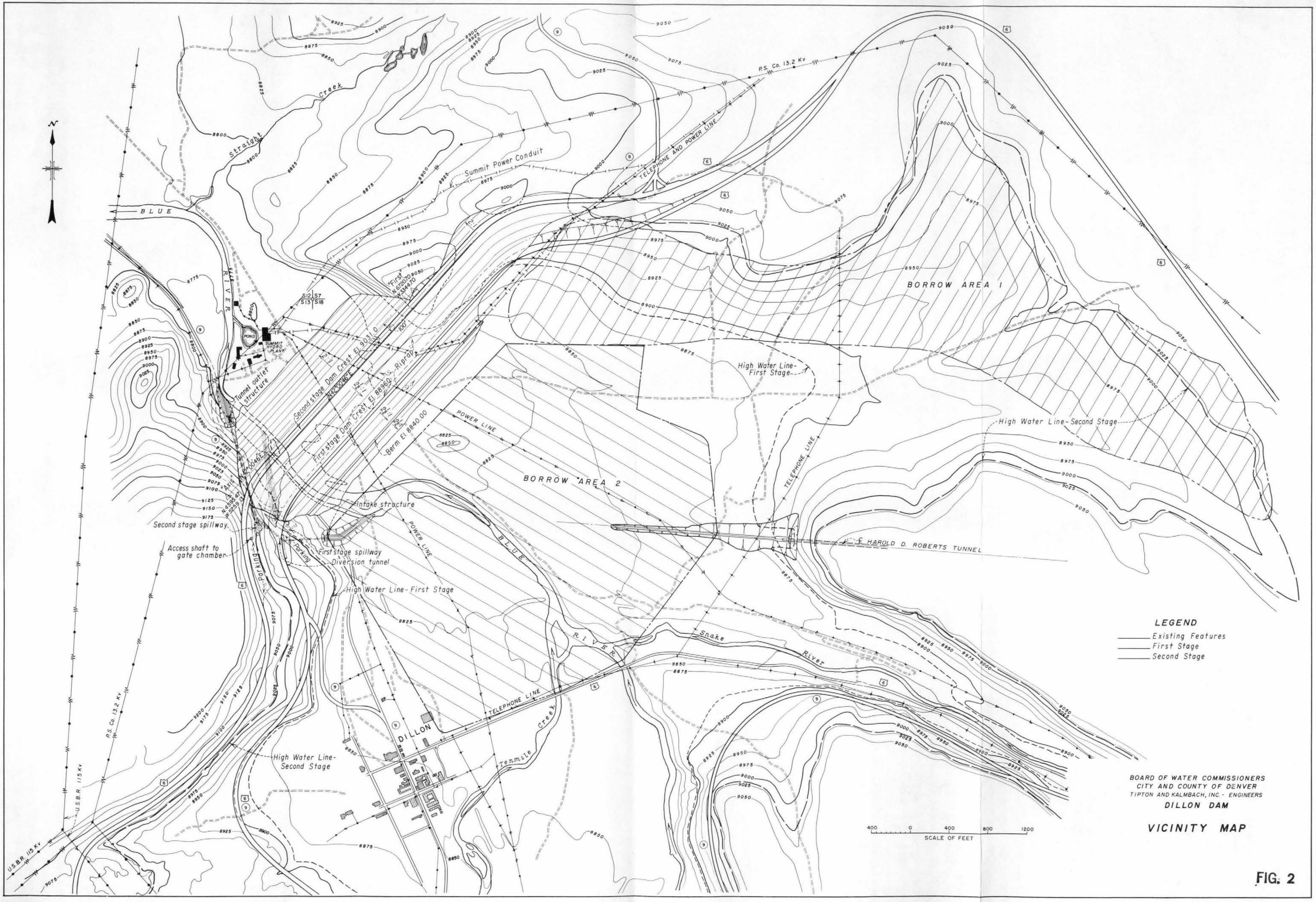
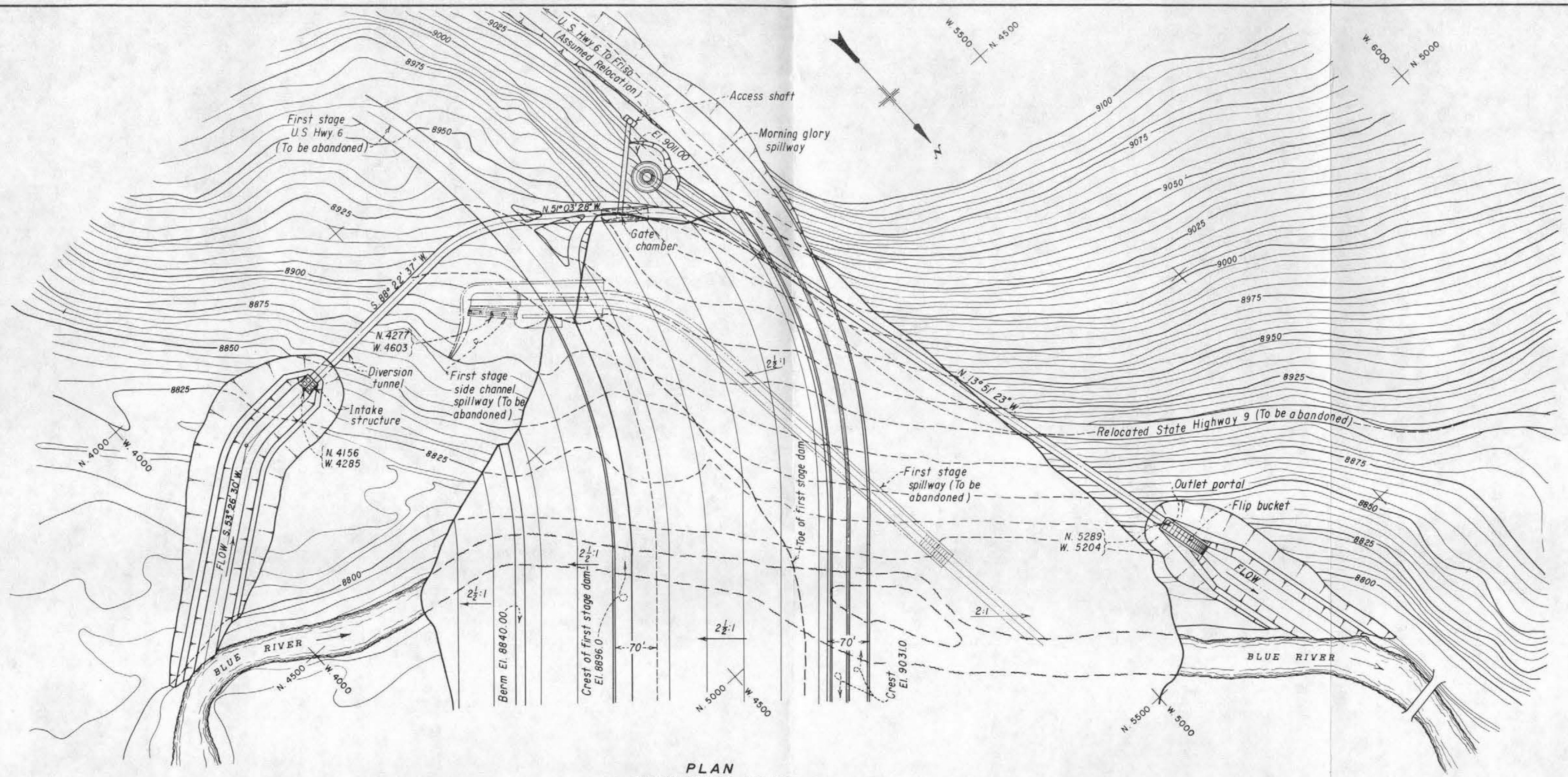

BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC. - ENGINEERS
DILLON DAM
VICINITY MAP
LEGEND
Existing Features
First Stage
Second Stage
SCALE OF FEET
BORROW AREA 1
BORROW AREA 2
BLUE RIVER
Straight Creek
Snake River
Tenmile Creek
DILLON
Summit Power Conduit
TELEPHONE AND POWER LINE
P.S. Co. 13.2 Kv
U.S.B.R. 115 Kv
POWER LINE
TELEPHONE LINE
SUMMIT HYDRO PLANT
POND
Tunnel outlet structure
Second stage Dam Crest El 9031.0
First stage Dam Crest El 8896.0
Berm El 8840.00
Riprap
Intake structure
Second stage spillway
Access shaft to gate chamber
First stage spillway
Diversion tunnel
Parking
High Water Line-First Stage
High Water Line-Second Stage
℄ HAROLD D. ROBERTS TUNNEL

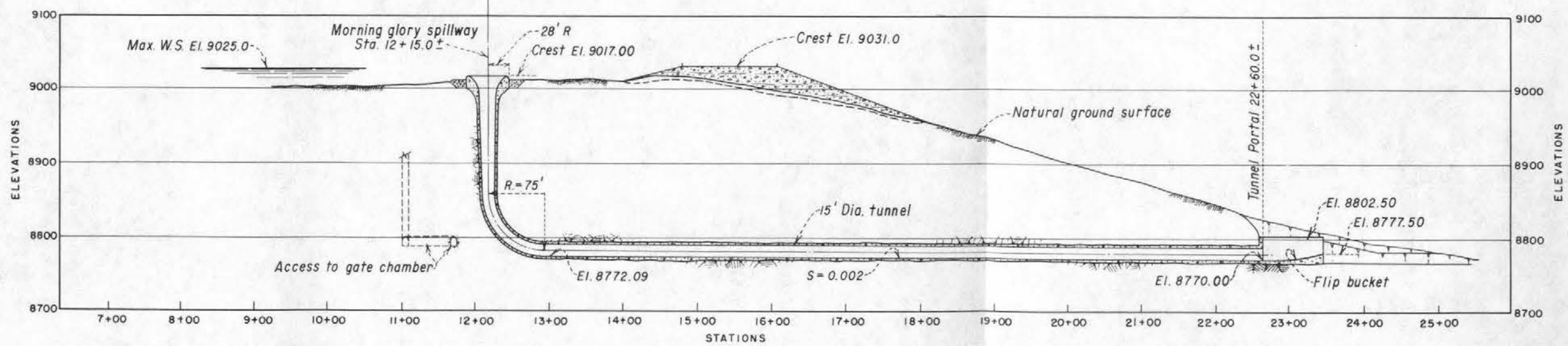
FIG. 2



PLAN

ALTERNATIVES 2A AND 2B

SCALE OF FEET



PROFILE - SECOND STAGE SPILLWAY

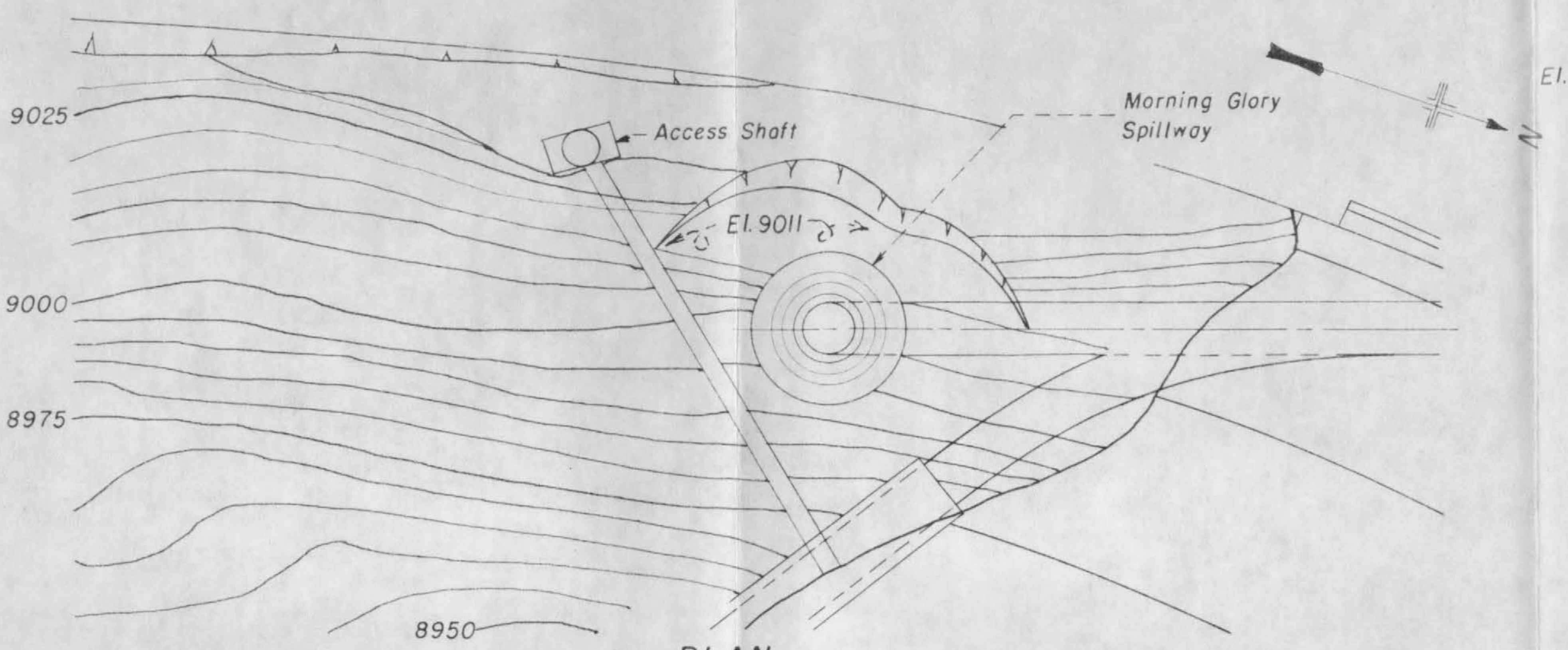
BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC. - ENGINEERS

DILLON DAM

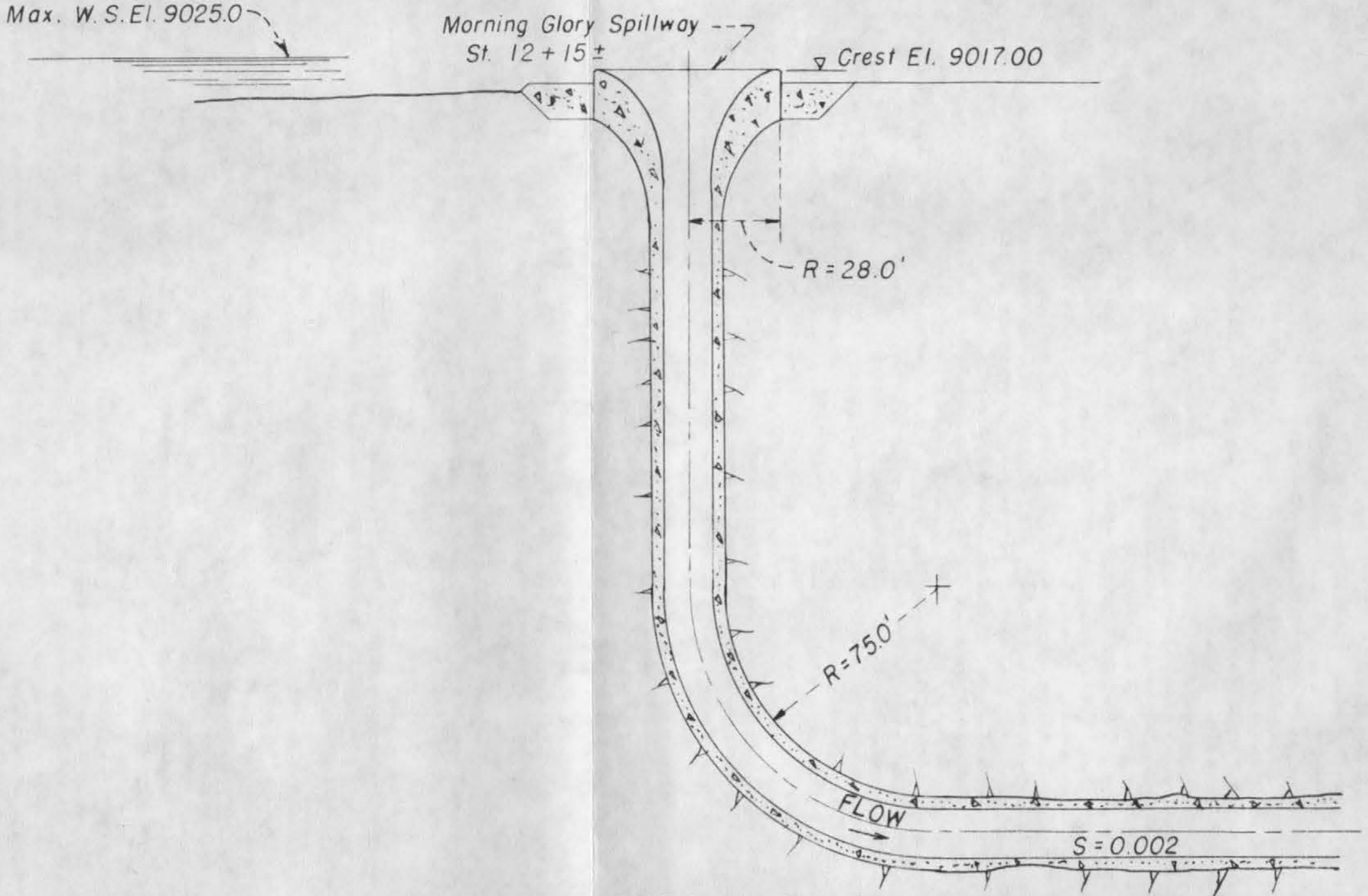
SECOND STAGE SPILLWAY

PLAN AND PROFILE

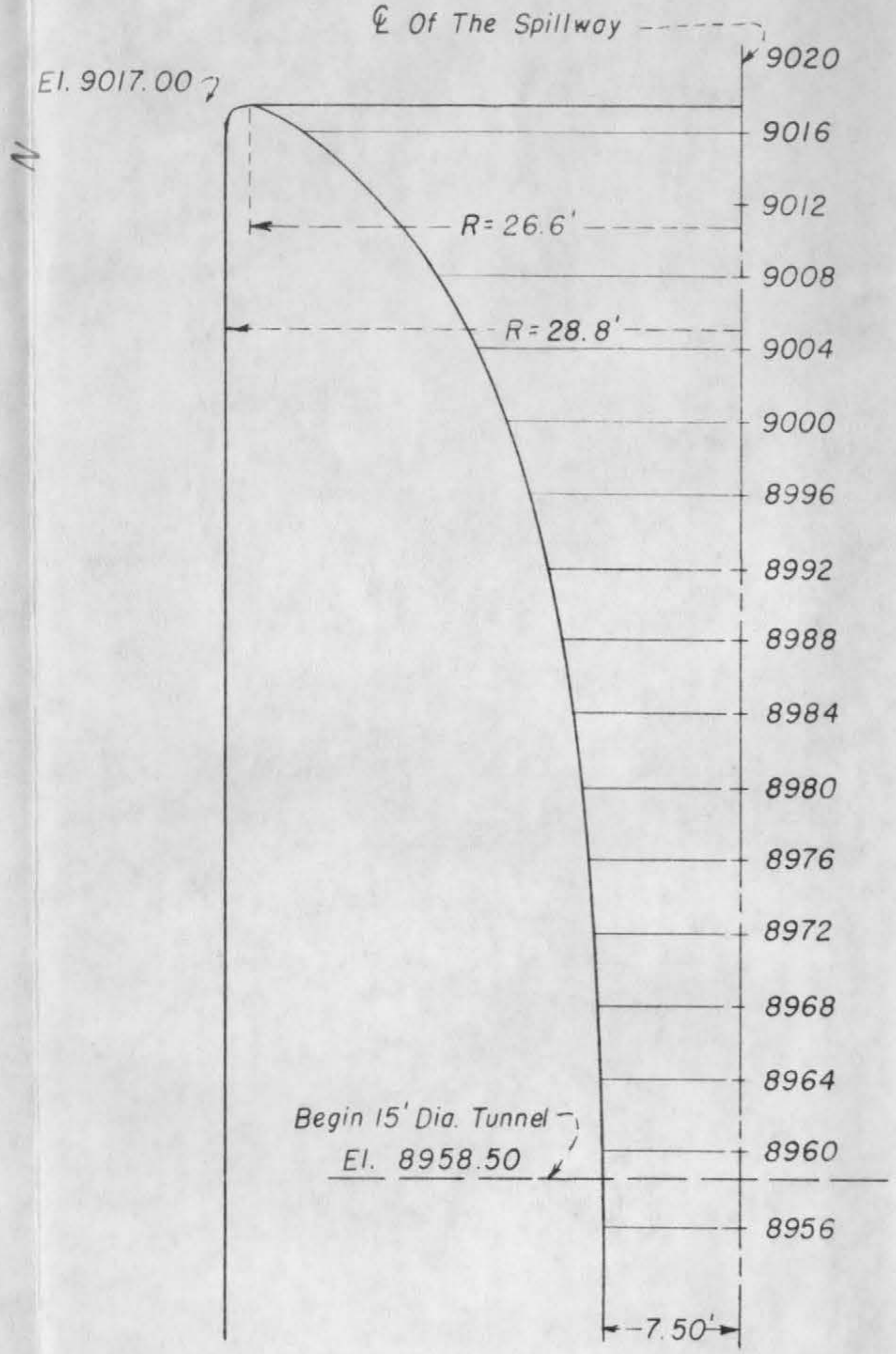
FIG. 3



PLAN



PROFILE OF SPILLWAY



SPILLWAY CREST DETAILS

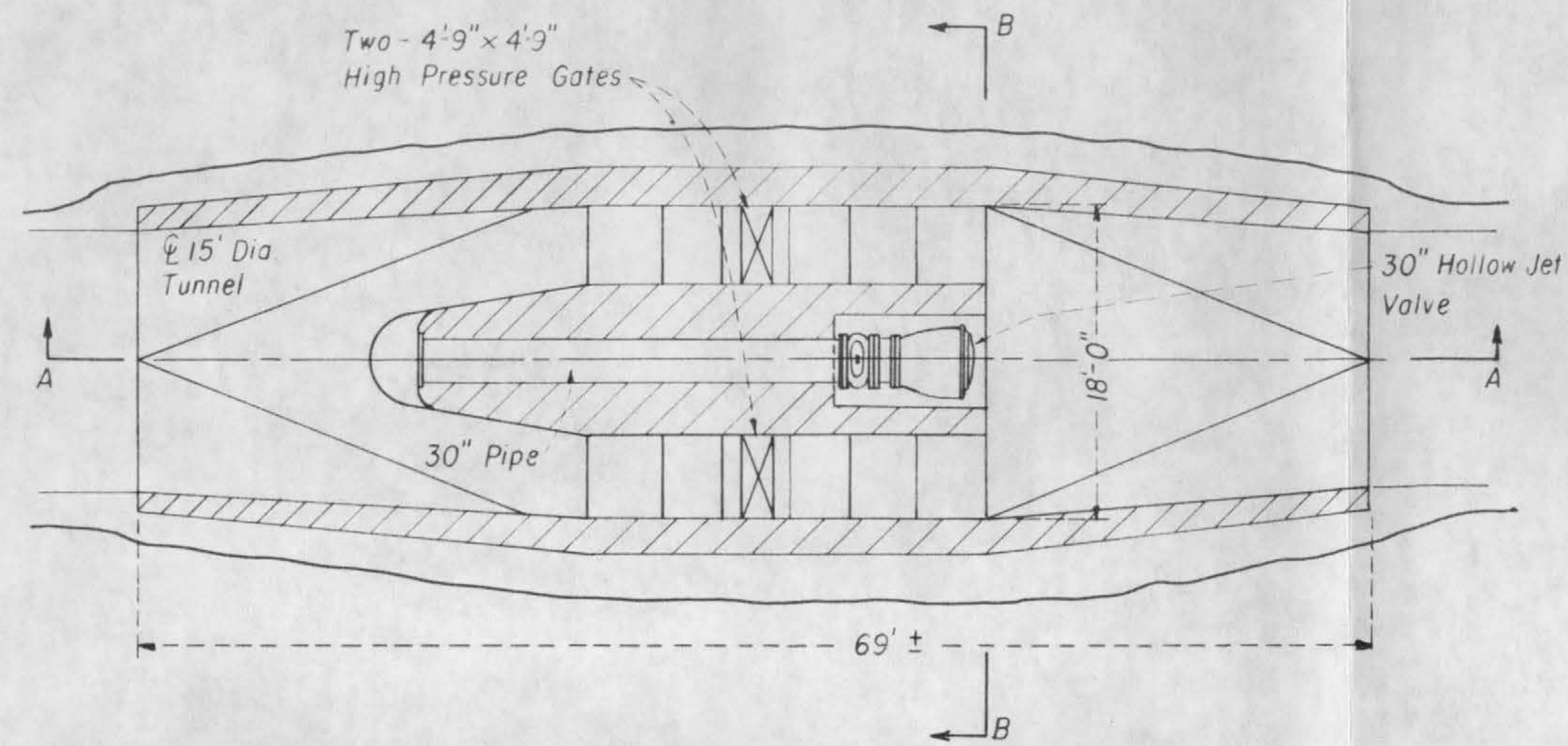
ELEVATION	RADIUS
9017.00	26.60
9016.96	26.23
9016.90	25.88
9016.82	25.52
9016.71	25.17
9016.58	24.81
9016.43	24.46
9016.36	24.32
9016.22	24.04
9015.94	23.54
9015.65	23.10
9014.94	22.15
9013.53	20.65
9010.70	18.31
9007.87	16.42
9005.30	14.87
9002.20	13.47
8998.66	11.96
8995.12	10.81
8991.58	10.01
8988.04	9.45
8984.50	9.01
8980.96	8.64
8977.42	8.32
8973.88	8.09
8968.00	7.82
8958.50	7.50

COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION
MODEL STUDY
DILLON DAM SPILLWAY

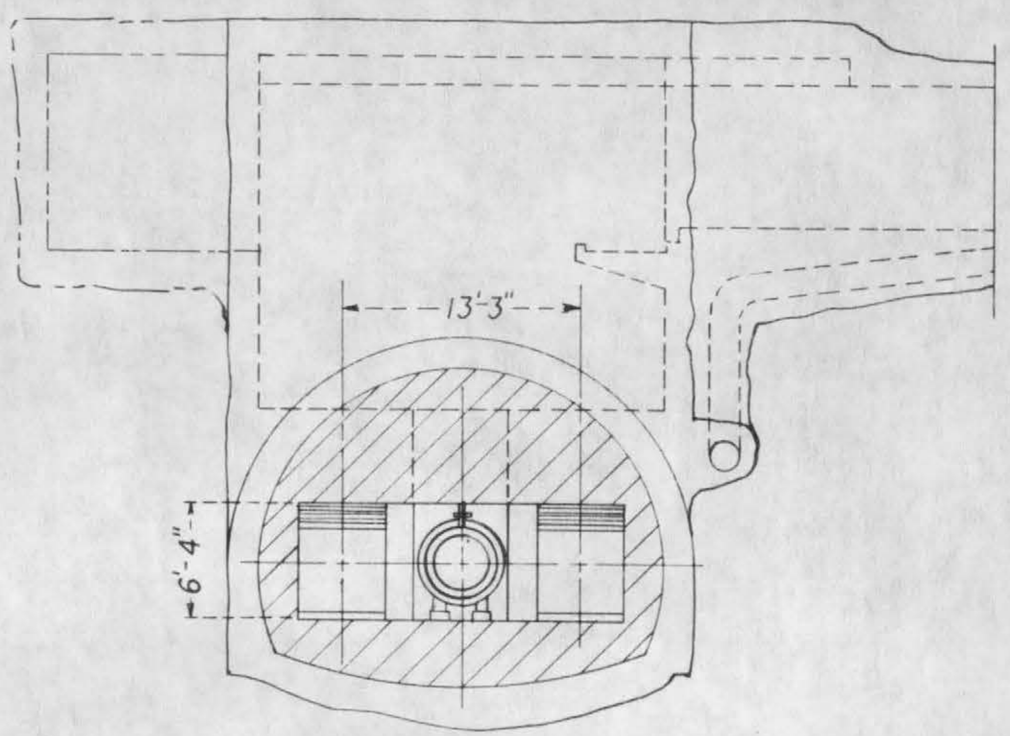
PRELIMINARY DESIGN OF THE MORNING GLORY SPILLWAY

Drawn by — A. A. El.
Traced by — R.P.M. M.C.C.
Checked by — S. K.
Approved S. Karaki

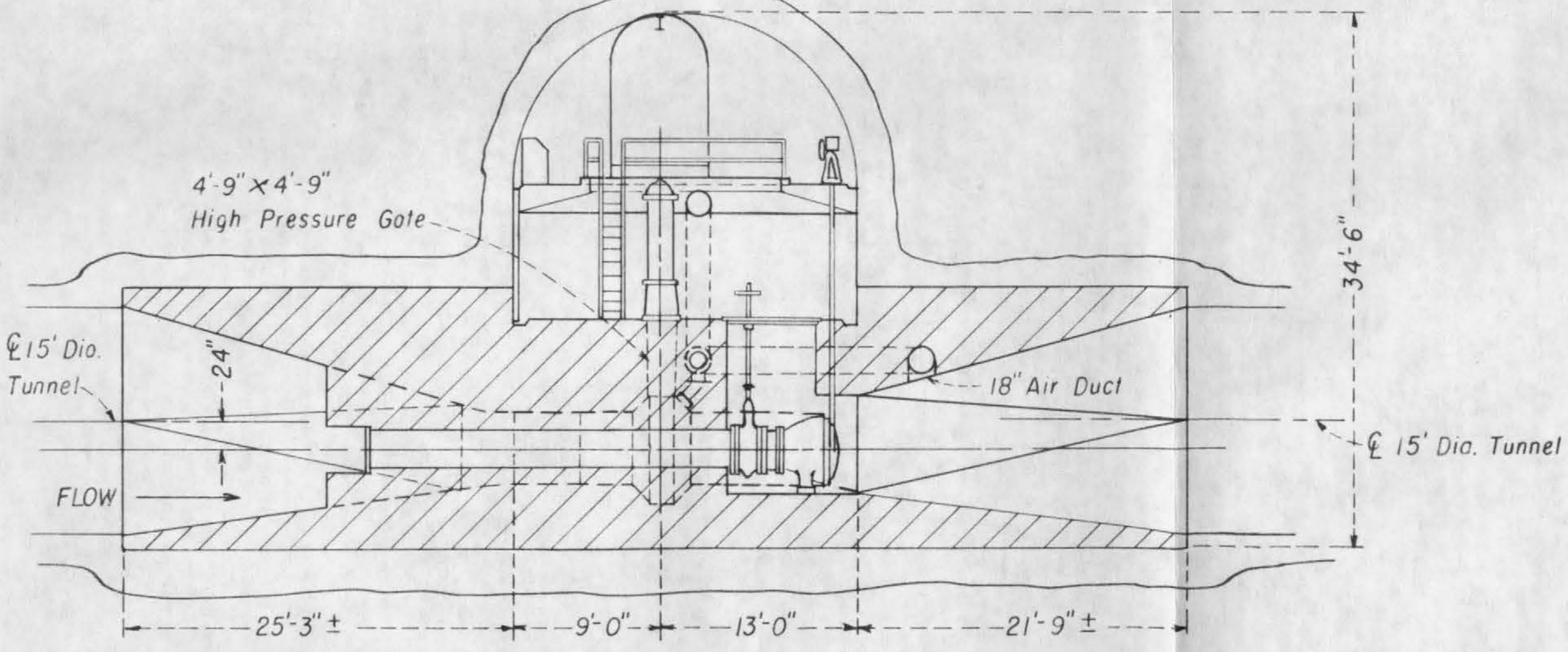
2002 | JUNE 1959 | FIG. 4



PLAN AT ℄ OF GATES



SECTION B-B



SECTION A-A

5 0 15 ft.
SCALE

NOTES

This drawing was made from dimensions shown on the preliminary drawing. Not all dimensions are included. Dimensions are in terms of prototype.

Gates were not modelled; Only the velocity and quantity of discharge were assimilated.

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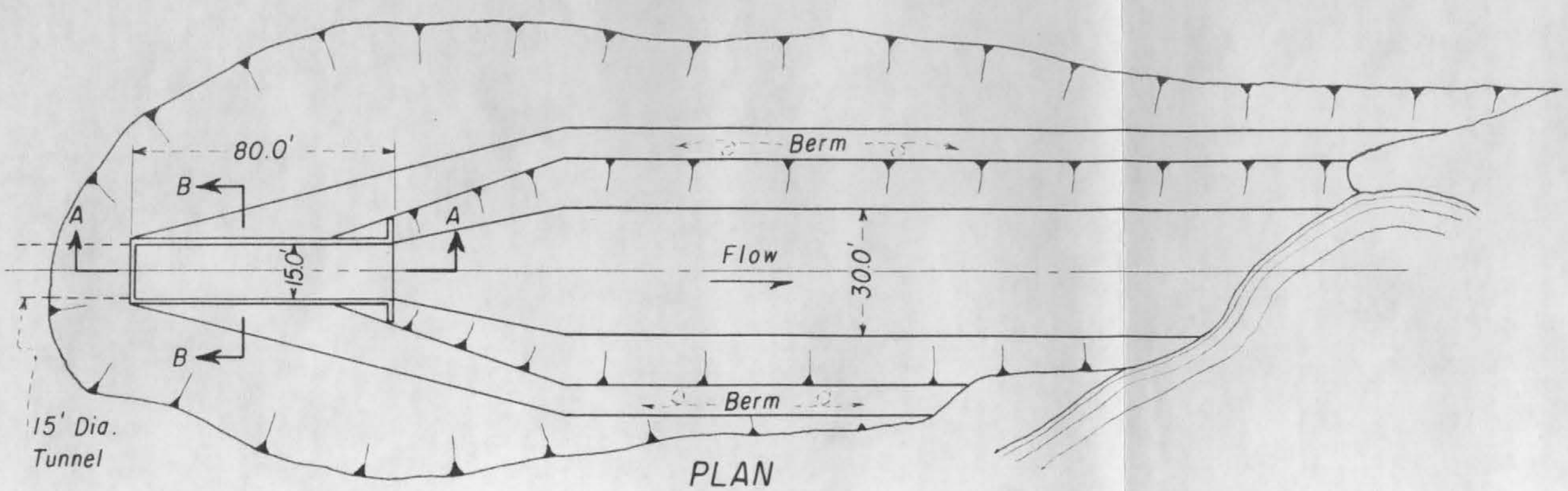
MODEL STUDY
DILLON DAM SPILLWAY

PRELIMINARY DESIGN
OF OUTLET WORKS

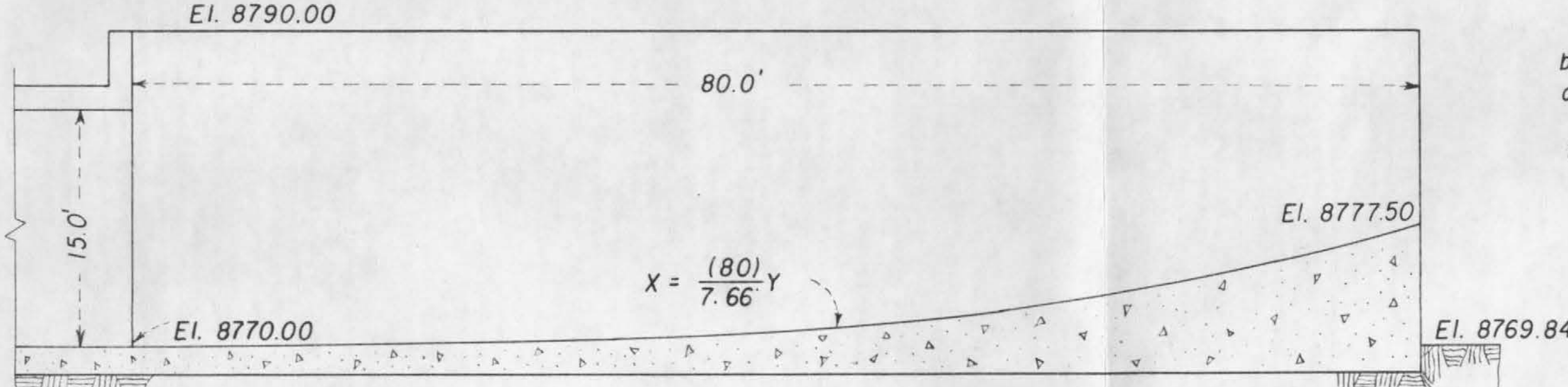
Drawn by R. P. M. Traced by R.P.M. M.C.G. Checked by S. K.	Approved S. Karaki

2002	JUNE 1959	FIG. 5

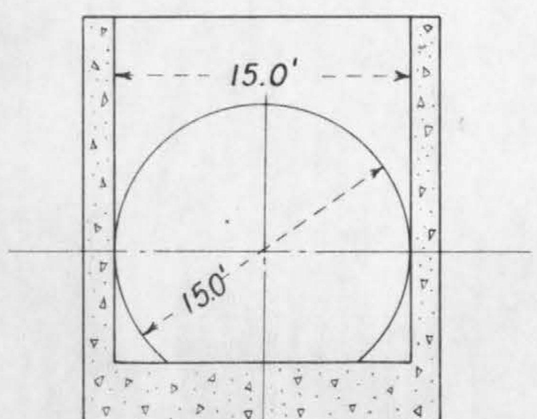


PLAN
DEFLECTOR BUCKET A

20 0 100 ft
SCALE



SECTION A-A



SECTION B-B

5 0 20 ft
SCALE

NOTES

The preliminary design of the deflector bucket was based on a cubical curve with a small exit angle at the lip.

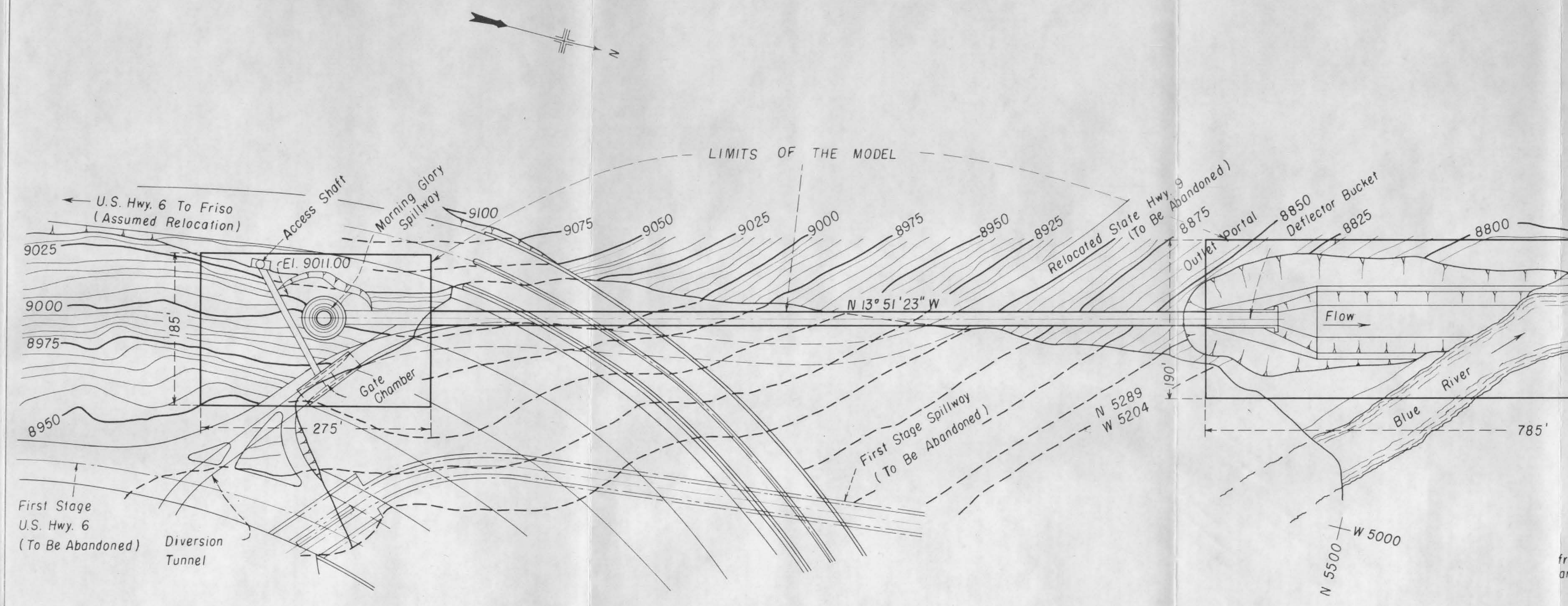
Dimensions shown are in terms of prototype. Structural details are not intended.

COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION
MODEL STUDY
DILLON DAM SPILLWAY

PRELIMINARY DESIGN - DEFLECTOR BUCKET A

Drawn by B. H. B. Traced by H.A. M.C.C. Checked by S. K.	Approved S. Karaki

2002	JUNE 1959	FIG. 6



NOTE

The plan of the spillway was traced from the preliminary design made by Tipton and Kalmbach Inc.

COLORADO STATE UNIVERSITY
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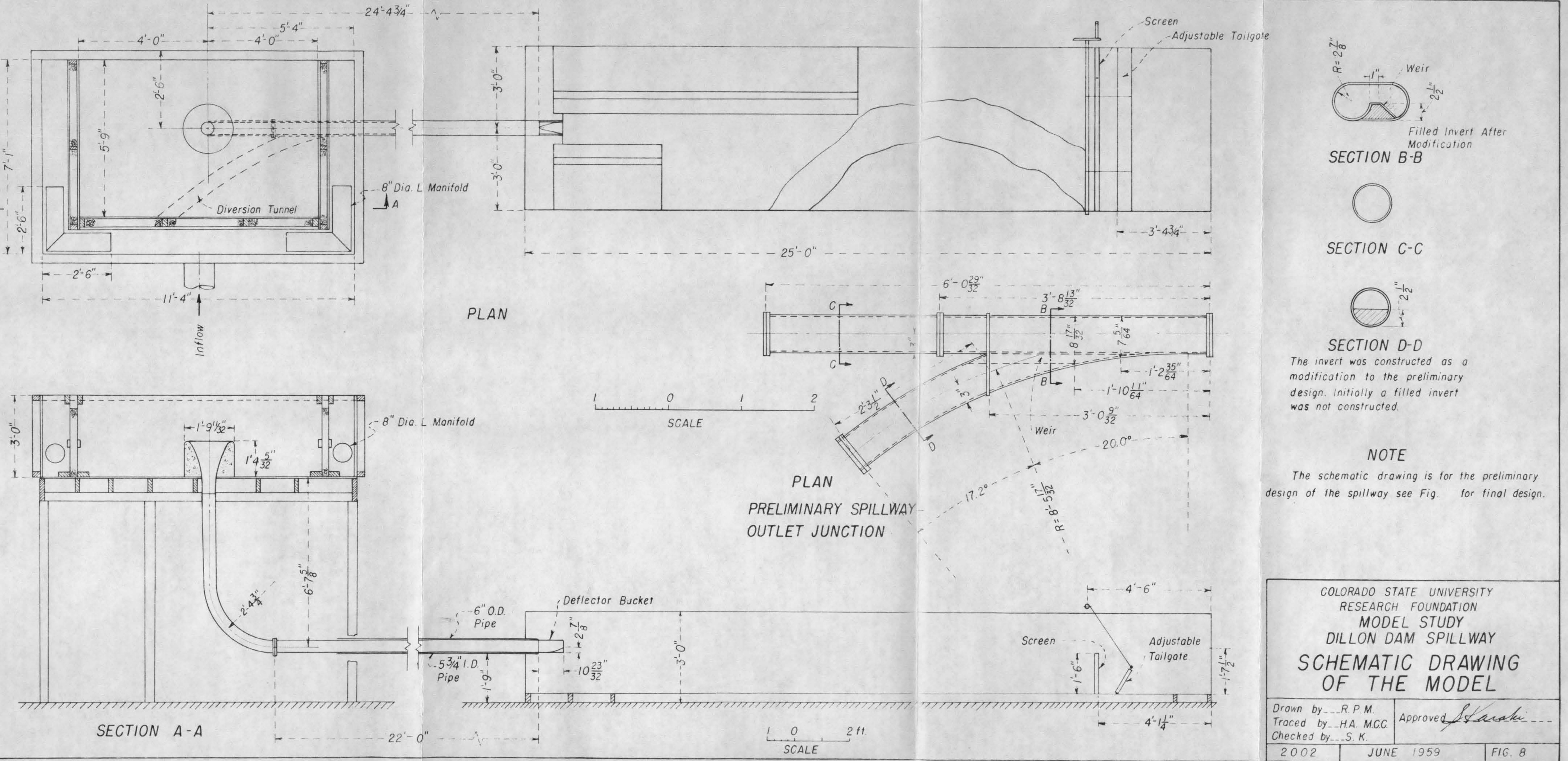
MODEL STUDY
DILLON DAM SPILLWAY

MODEL LIMITS

Drawn by R.P.M. Traced by S.A.M.C.C. Checked by S.K.	Approved [signature]

2002	JUNE 1959	FIG. 7



The invert was constructed as a modification to the preliminary design. Initially a filled invert was not constructed.

NOTE

The schematic drawing is for the preliminary design of the spillway see Fig. for final design.

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MODEL STUDY
DILLON DAM SPILLWAY

SCHEMATIC DRAWING OF THE MODEL

Drawn by R. P. M.
Traced by H.A. M.C.C.
Checked by S. K.
Approved S. Karaki

2002 | JUNE 1959 | FIG. 8

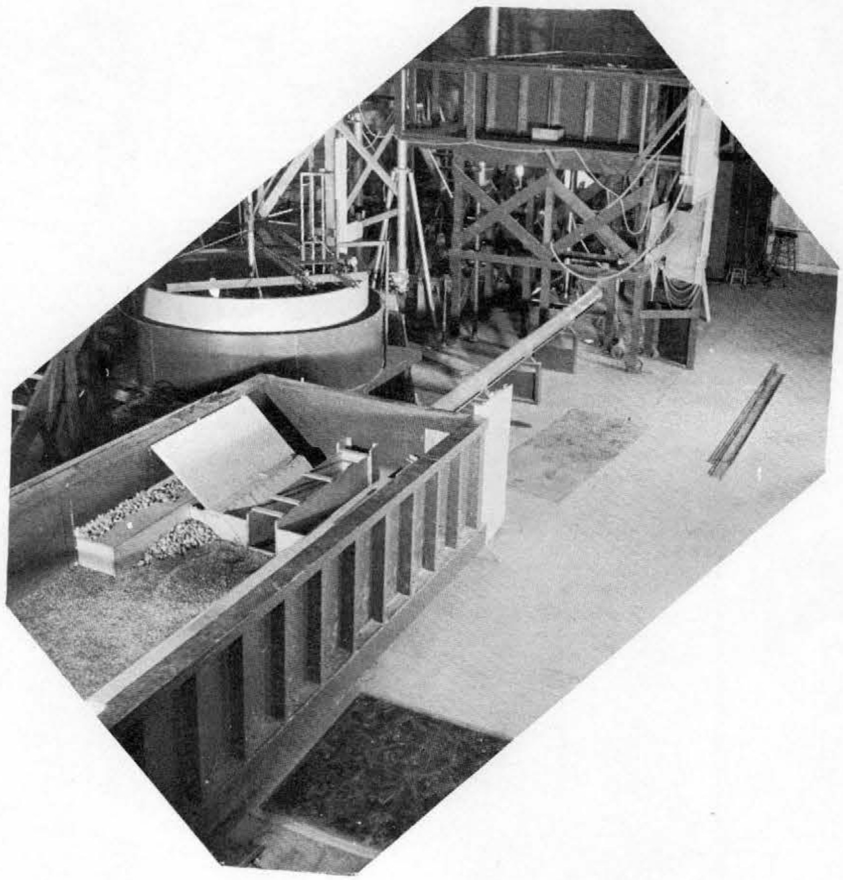


Fig. 9. Photograph of the Completed Model.

Fig. 10. Construction of Morning Glory Spillway. Installation of Piezometers.

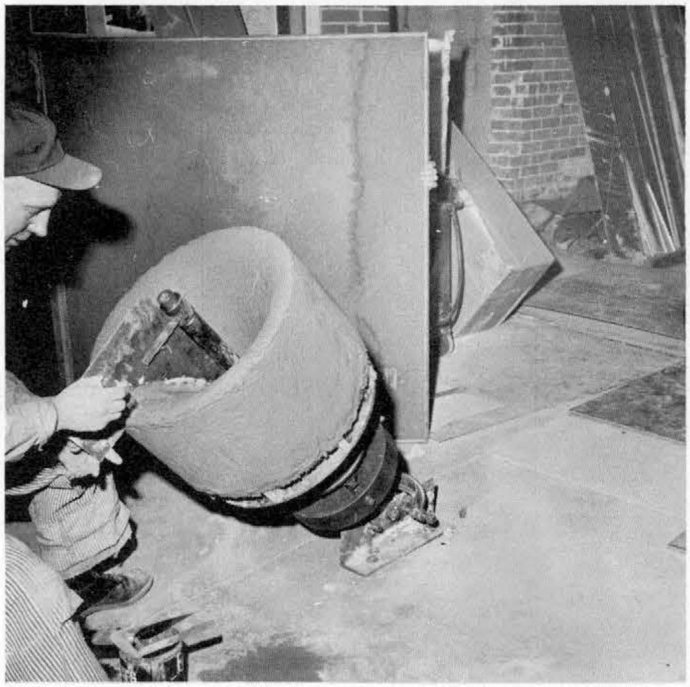


Fig. 11. Construction of Morning Glory Spillway Placing and screeding the mortar by template.

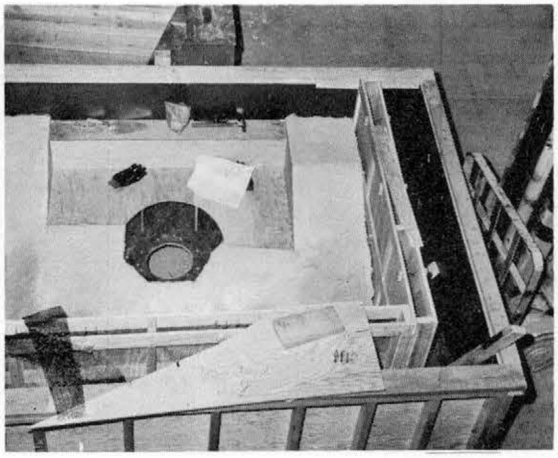


Fig. 12. Construction of Topography in the Crest Area.

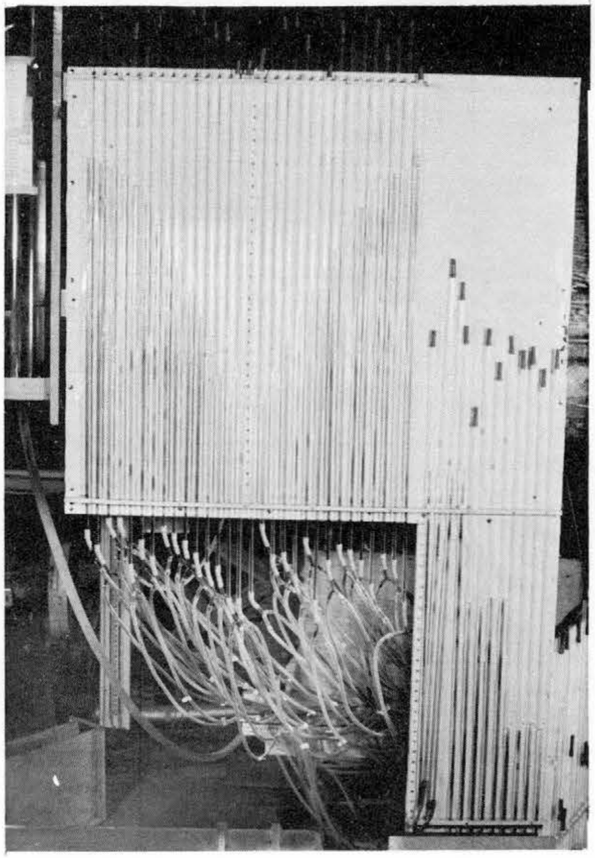
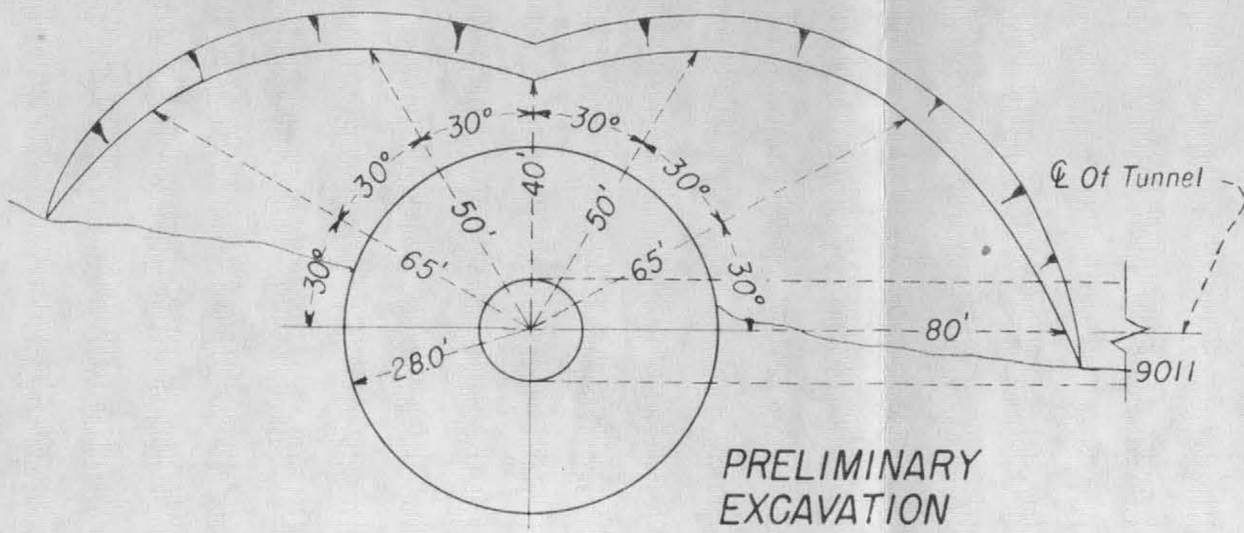
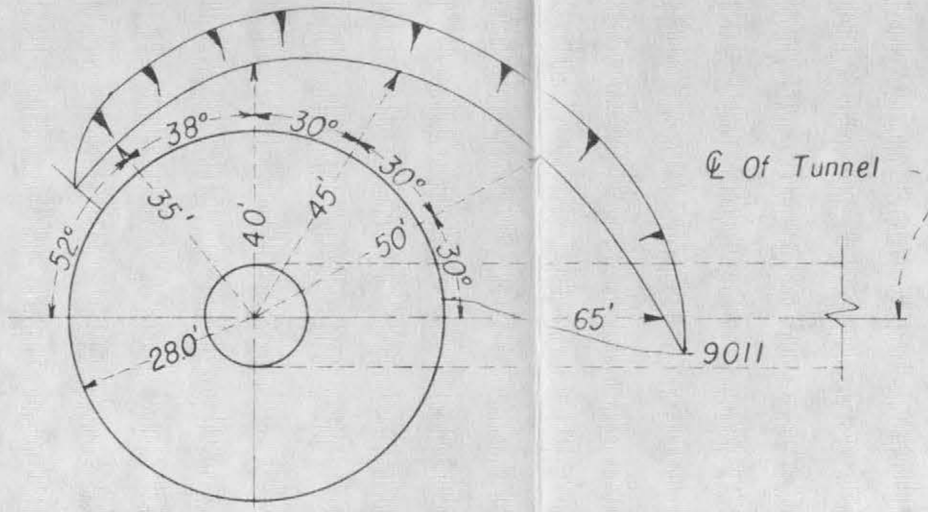


Fig. 13. The Manometer Board for Measuring Crest Pressures.

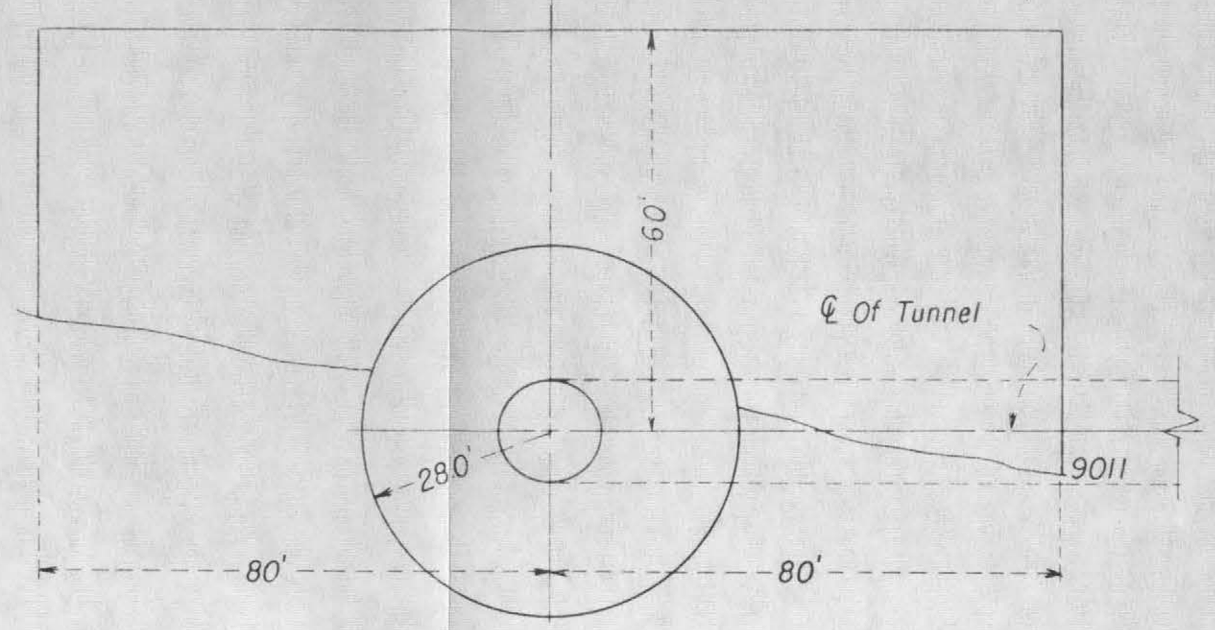
Fig. 14. Construction of the Exterior Mold for the Vertical Bend of the Spillway Shaft.



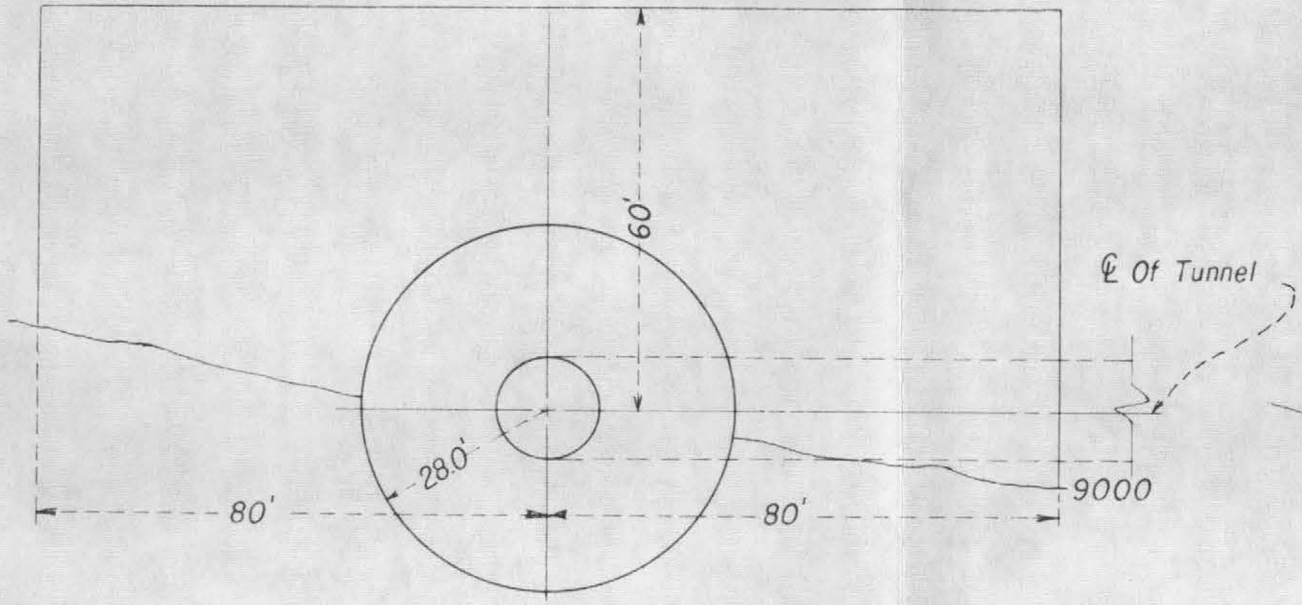
PRELIMINARY EXCAVATION



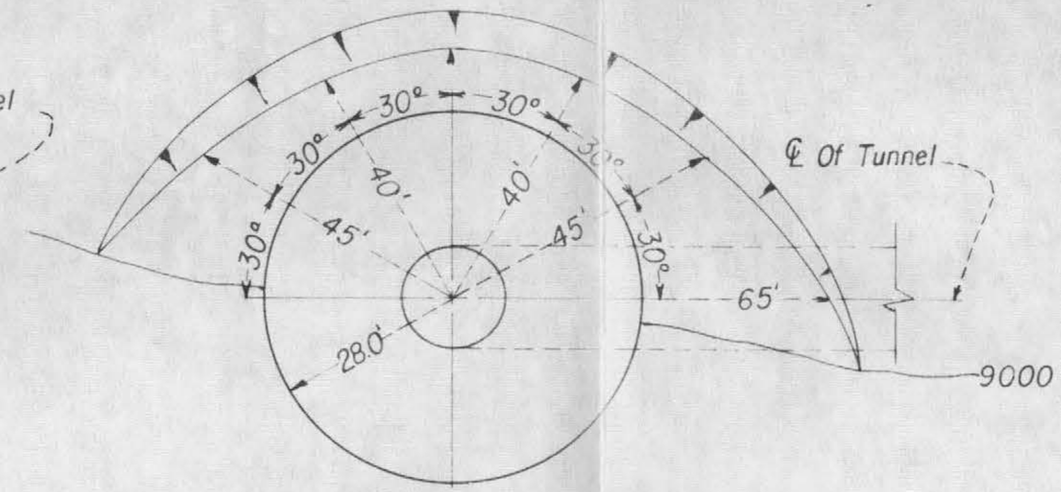
MODIFIED EXCAVATION NO. 1



MODIFIED EXCAVATION NO. 2



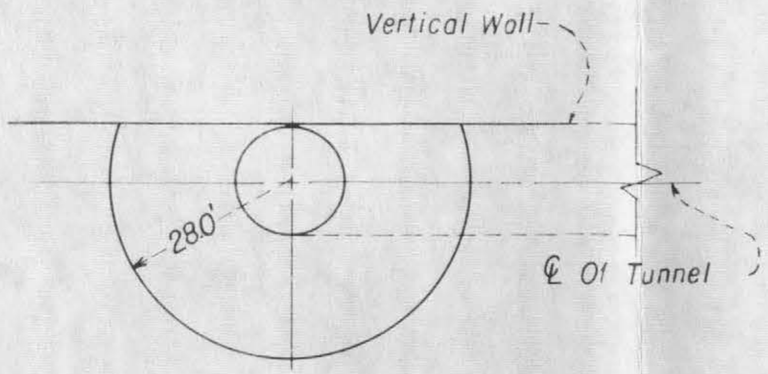
MODIFIED EXCAVATION NO. 3



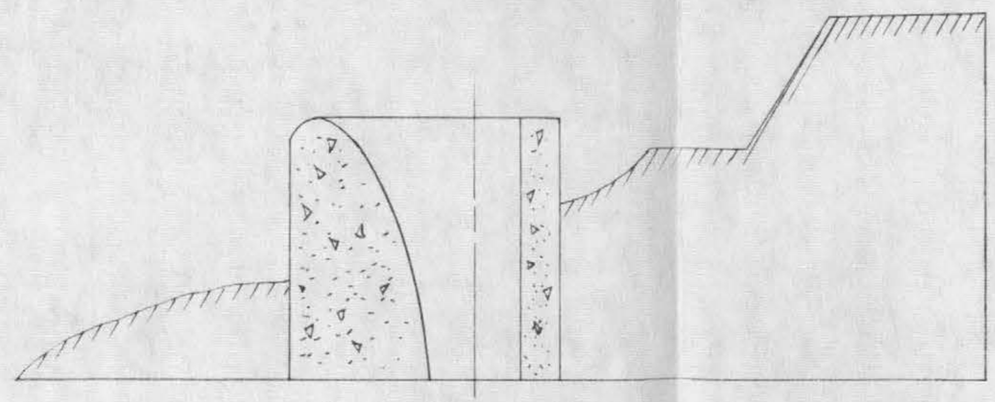
MODIFIED EXCAVATION NO. 4

NOTE

Dimensions for the excavation are approximate to the nearest 5 ft. See Figs. 54 and 56 for the relocated and final crest excavation studies.



MODIFIED EXCAVATION NO. 5



SECTION A-A

SCALE

COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION

MODEL STUDY
DILLON DAM SPILLWAY

EXCAVATION STUDIES
PRELIMINARY CREST LOCATION

Drawn by __ R.P.M. Traced by __ R.P.M. M.C.C. Checked by __ S. K.	Approved S. Karaki

2002	JUNE 1959	FIG. 15

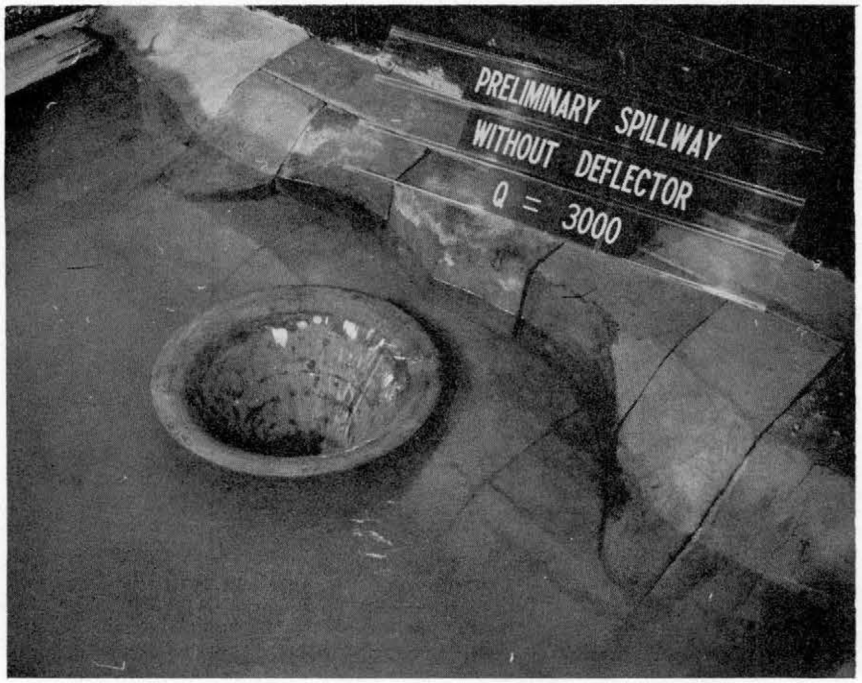


Fig. 16. Preliminary Spillway Location and Preliminary Excavation. Q = 3000 c.f.s.

Fig. 17. Preliminary Spillway Location and Preliminary Excavation. Q = 5000 c.f.s. Note formation of fin opposite the point in the excavation.

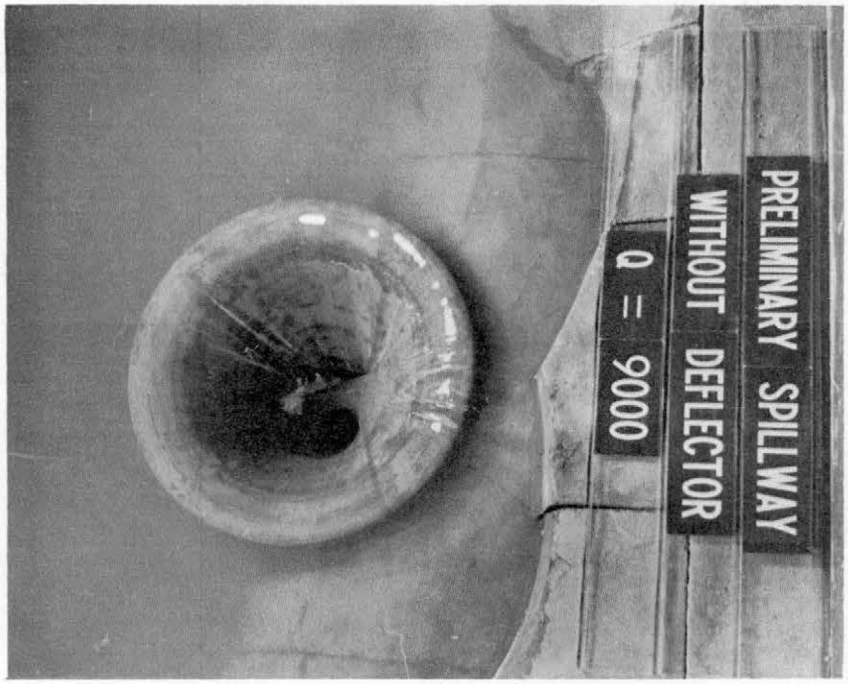


Fig. 18. Preliminary Spillway Location and Preliminary Excavation. Q = 9000 c.f.s. The fin becomes very prominent for this discharge.

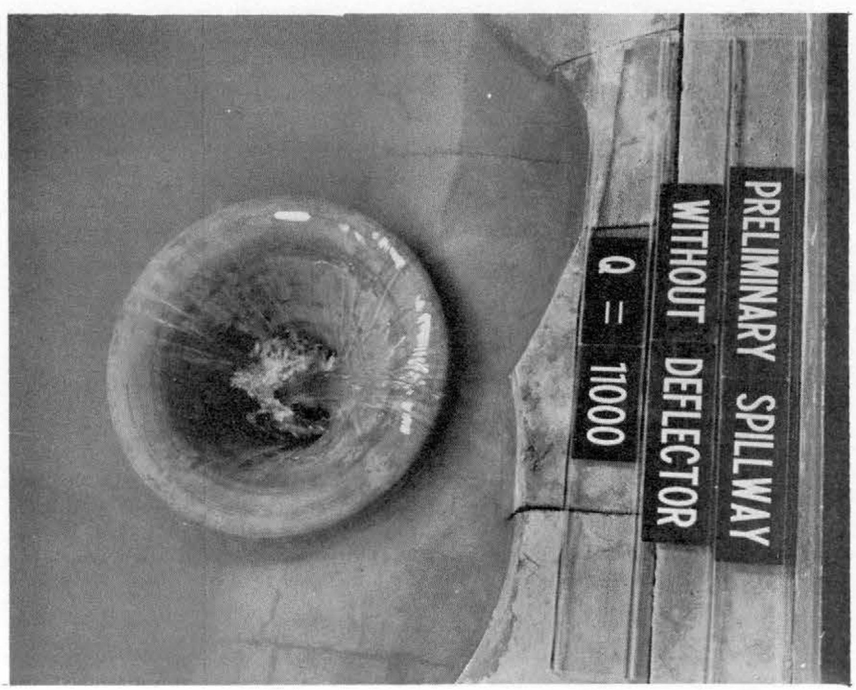


Fig. 19. Preliminary Spillway Location and Preliminary Excavation. Q = 11,000 c.f.s. Note formation of boil in throat of spillway shaft.

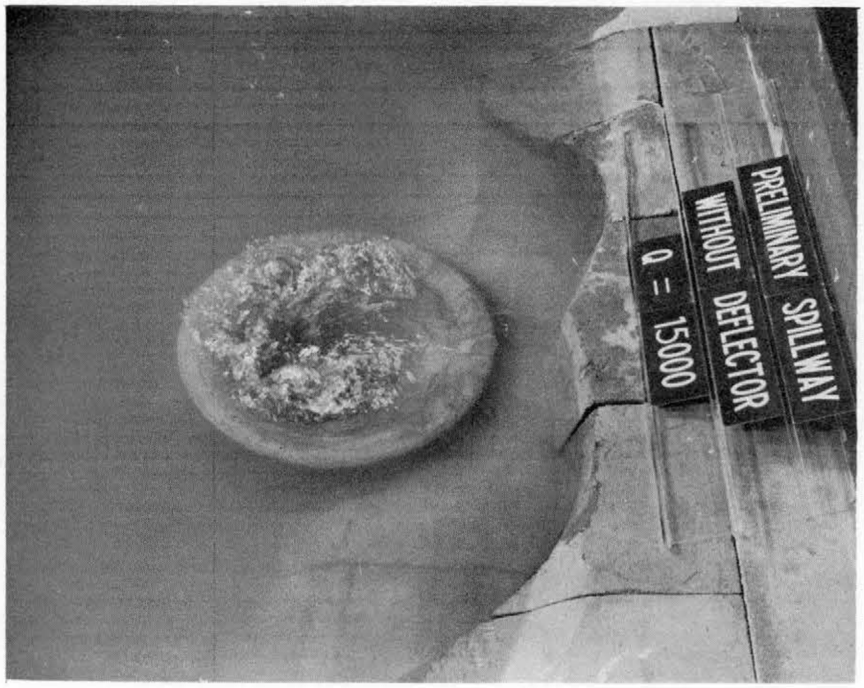


Fig. 20. Preliminary Spillway Location and Preliminary Excavation. Q = 15,000 c.f.s. Spillway is partially submerged.

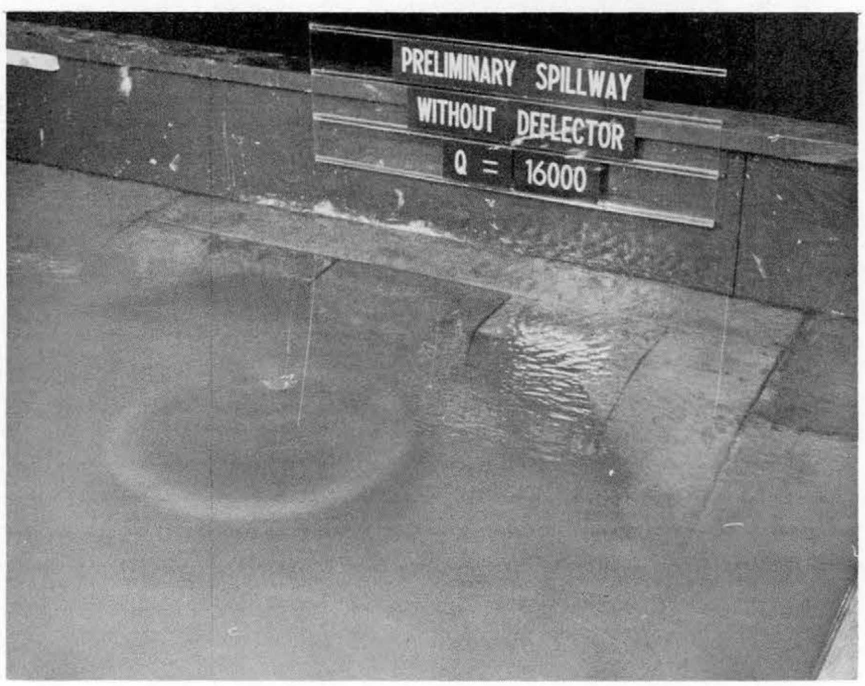
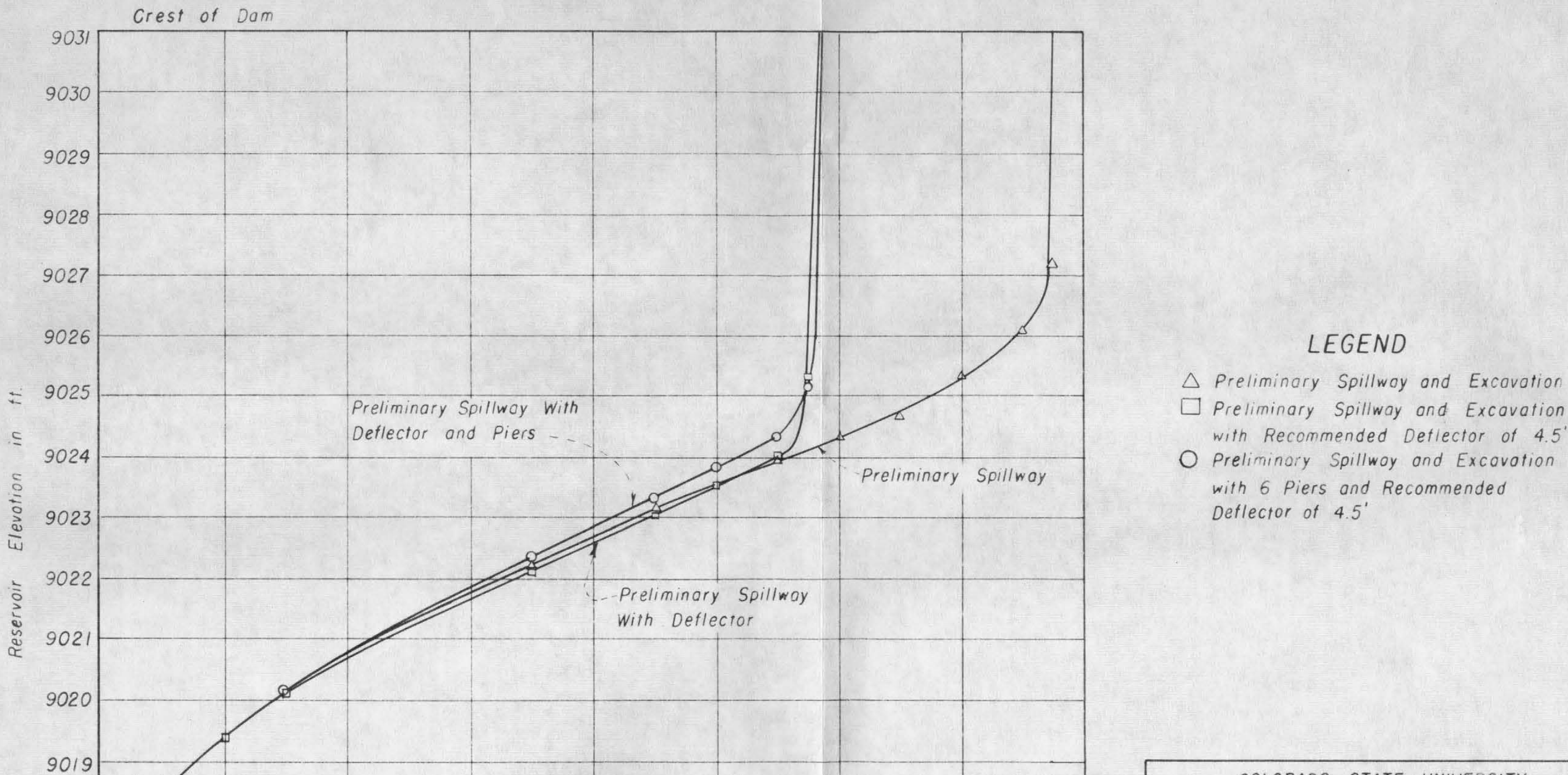


Fig. 21. Preliminary Spillway Location and Preliminary Excavation. Q = 16,000 c.f.s. Spillway is completely submerged.



COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION
MODEL STUDY
DILLON DAM SPILLWAY

SPILLWAY RATING CURVES FOR PRELIMINARY LOCATION

Drawn by R. P. M. Traced by R.P.M. M.C.C. Checked by S. K.	Approved S. Karaki	
2002	JUNE 1959	FIG. 22

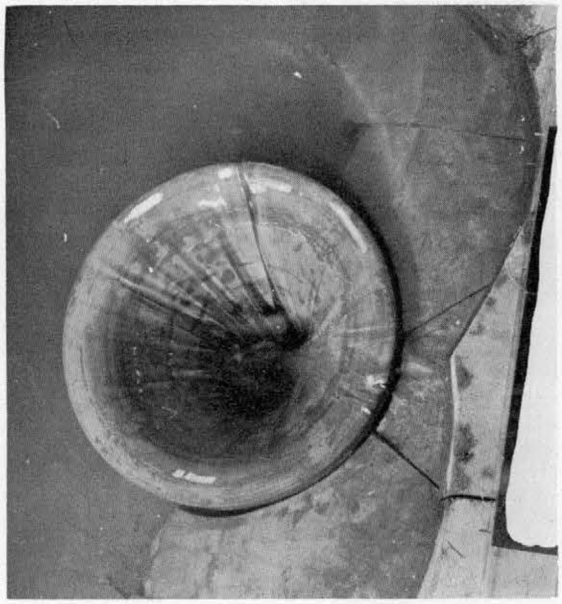


Fig. 23. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 7000 c.f.s.

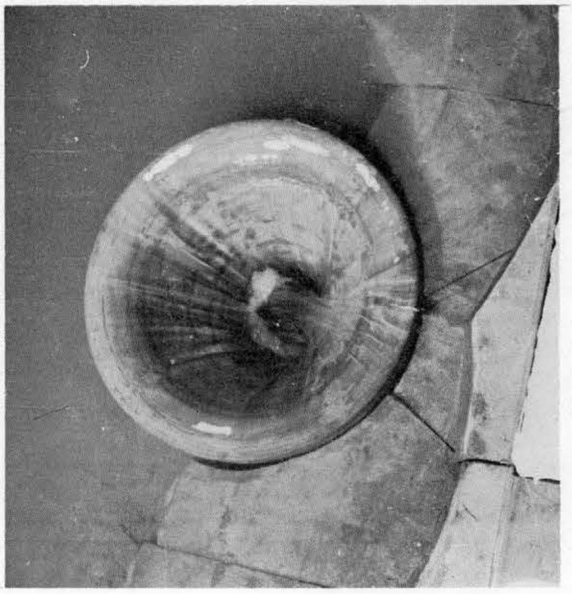


Fig. 24. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 9000 c.f.s.

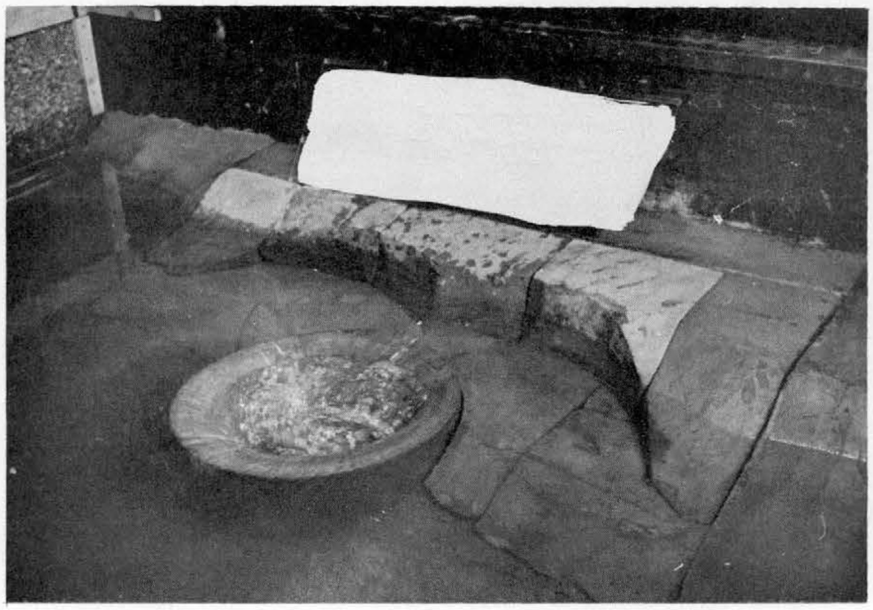
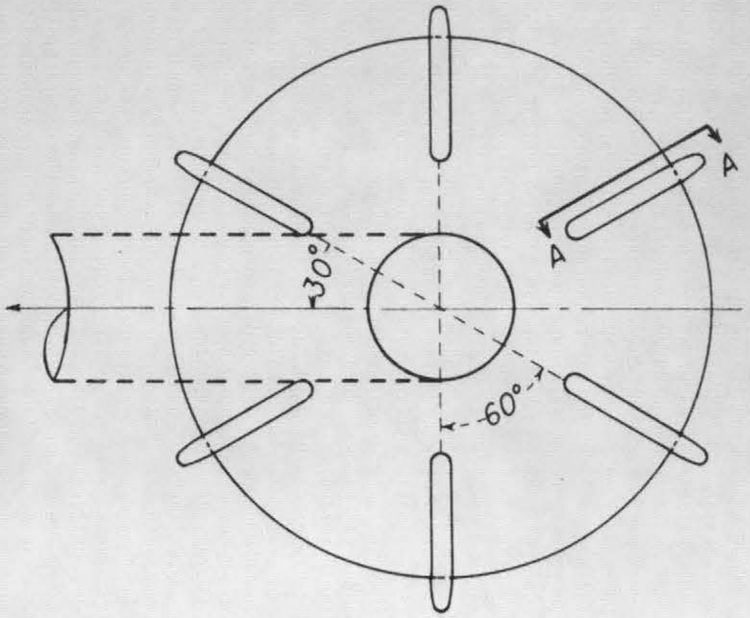
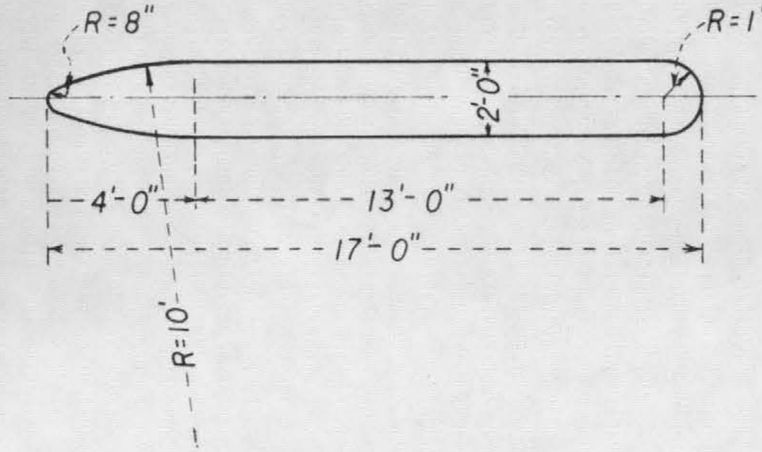


Fig. 25. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 11,000 c.f.s.

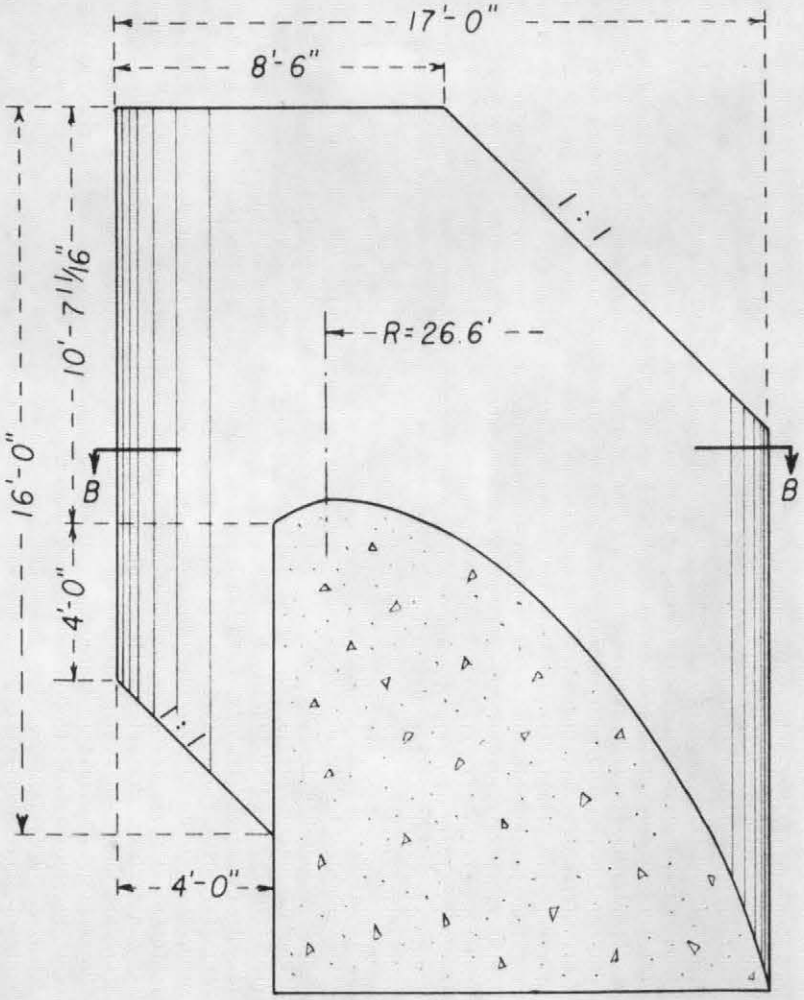
Fig. 26. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 11,500 c.f.s.



PLAN



SECTION B-B



SECTION A-A

NOTE

Number and orientation of piers shown for interim study. Final location of piers are shown in Fig. 56. Pier size is as shown on this drawing.

SCALE

COLORADO STATE UNIVERSITY RESEARCH FOUNDATION MODEL STUDY DILLON DAM SPILLWAY CREST PIERS PLAN AND SECTIONS		
Drawn by ___ M.C.C. Traced by __ H.A. M.C.C. Checked by __ S.K.	Approved S. Karaki	
2002	JUNE 1959	FIG. 27

Fig. 28. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 7000 c.f.s.

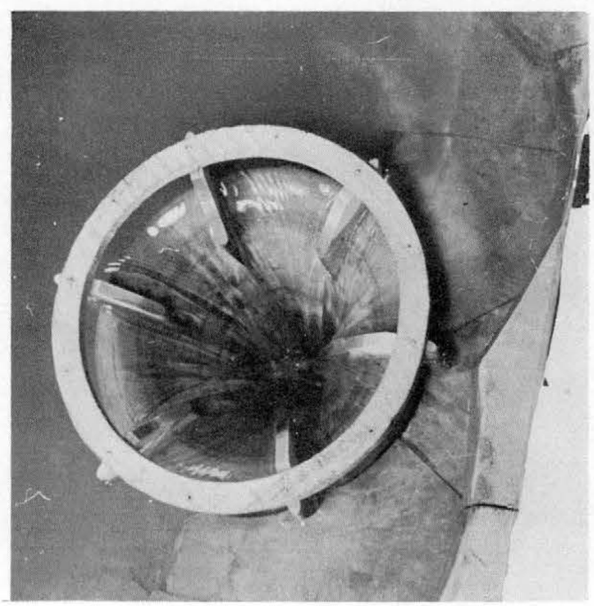


Fig. 29. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 9000 c.f.s.

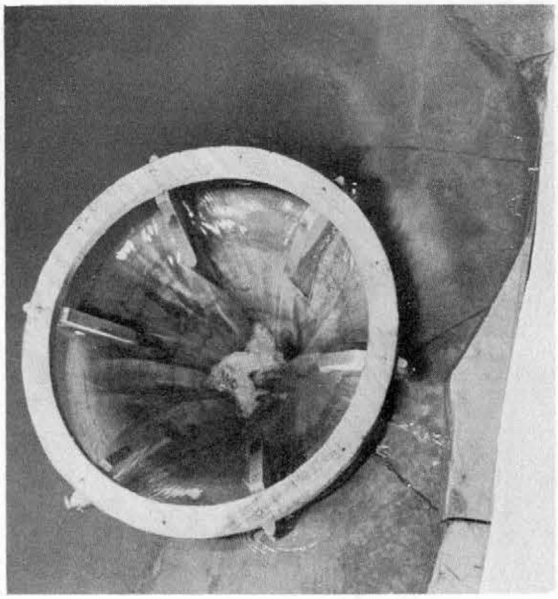


Fig. 30. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 11,000 c.f.s.

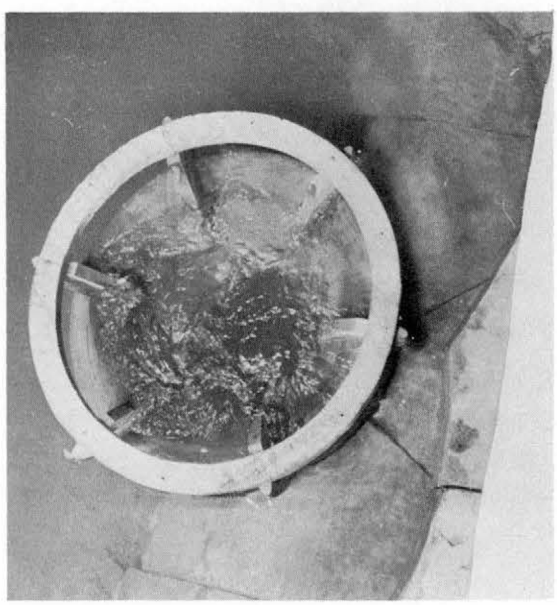


Fig. 31. Preliminary Spillway Location and Preliminary Excavation. Recommended deflector located at vertical bend. Q = 11,500 c.f.s.

Fig. 32. Preliminary Spillway Location and Excavation Modification No. 1. Recommended deflector located at vertical bend. Q = 7000 c.f.s.

Fig. 33. Preliminary Spillway Location and Excavation Modification No. 1. Recommended deflector located at vertical bend. Q = 9000 c.f.s.

Fig. 34. Preliminary Spillway Location and Excavation Modification No. 1. Recommended deflector located at vertical bend. Q = 11,000 c.f.s.

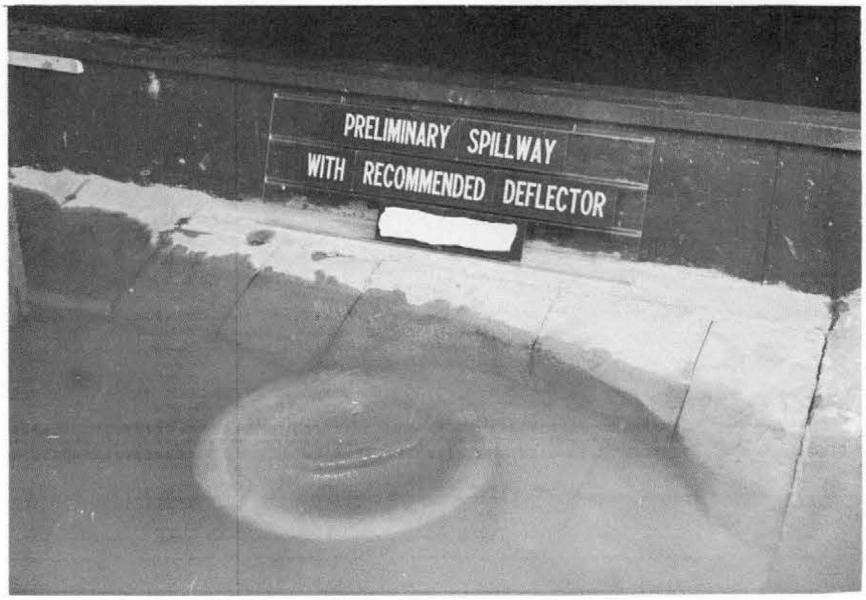


Fig. 35. Preliminary Spillway Location and Excavation Modification No. 1. Recommended deflector located at vertical bend. Q = 11,500 c.f.s.

Fig. 36. Spiral Flow in Vertical Shaft Caused by the Unsymmetrical Excavation of Modification No. 1 Q = 9000 c.f.s.

Fig. 37. Note Vortex Pocket in the Vertical Shaft. Q = 11,500 c.f.s.

Fig. 38. Preliminary Spillway Location and Excavation Modification No. 2. Recommended deflector located at vertical bend. Q = 7000 c.f.s.

Fig. 39. Preliminary Spillway Location and Excavation Modification No. 2. Recommended deflector located at vertical bend. Q = 9000 c.f.s.

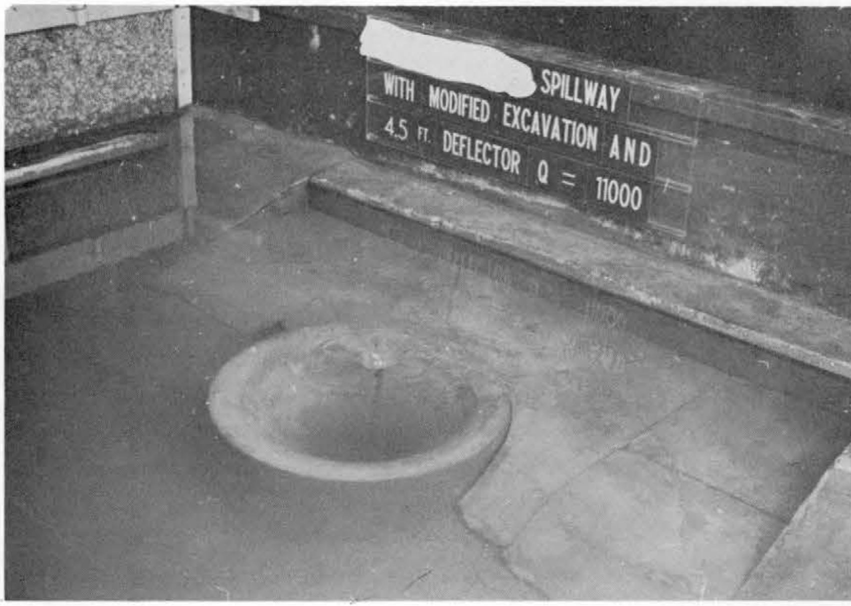


Fig. 40. Preliminary Spillway Location and Excavation Modification No. 2. Recommended deflector located at vertical bend. Q = 11,000 c.f.s.

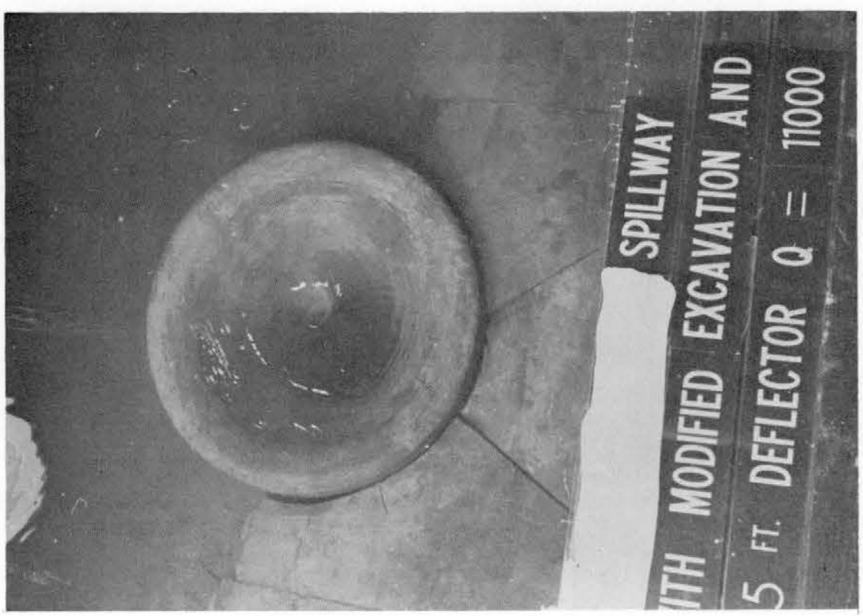


Fig. 41. Vertical View of Fig. 40. Q = 11,000 c.f.s.

Fig. 42. Preliminary Spillway Location Excavation Modification No. 2, with piers. Recommended deflector located at vertical bend. Q = 7000 c.f.s.

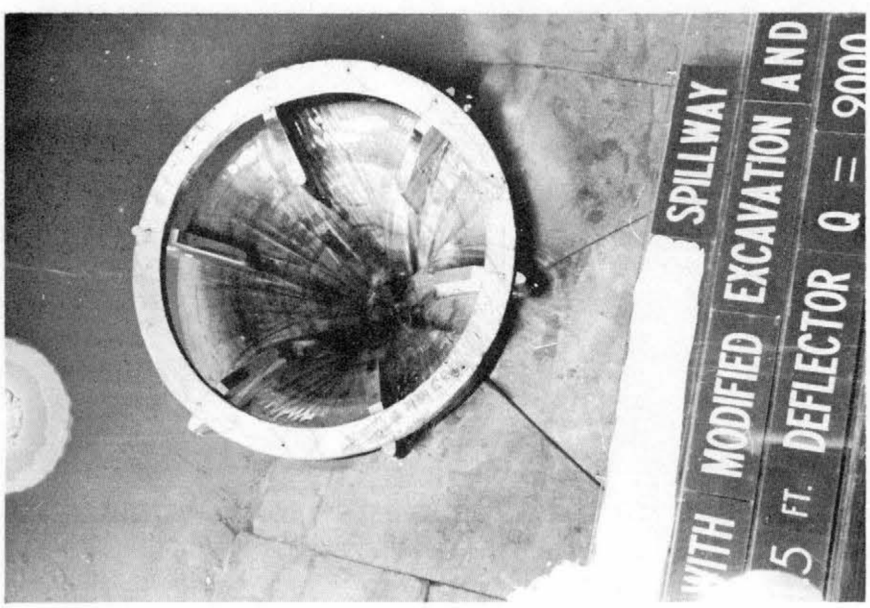


Fig. 43. Preliminary Spillway Location Excavation Modification No. 2, with Piers. Recommended deflector located at vertical bend. Q = 9000 c.f.s.

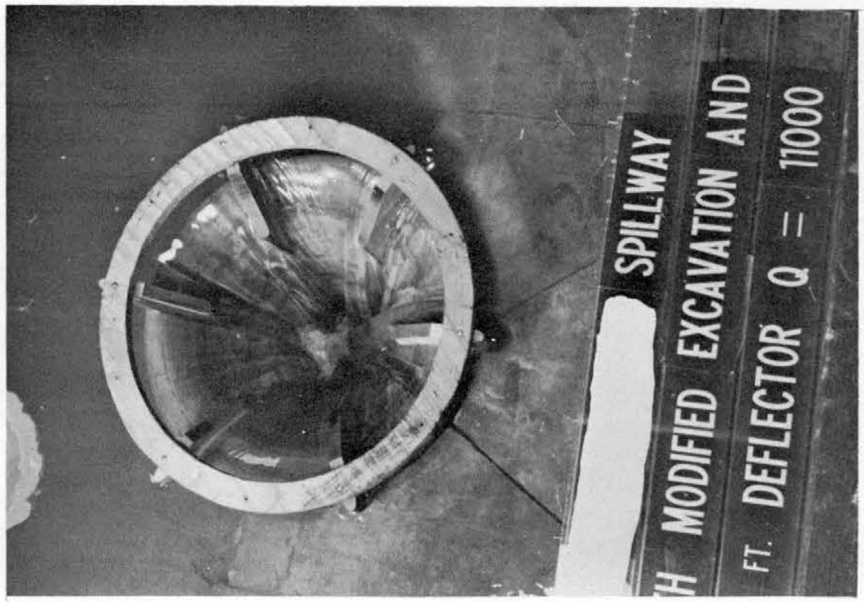


Fig. 44. Preliminary Spillway Location. Excavation Modification No. 2, with Piers. Recommended deflector located at vertical bend. Q = 11,000 c.f.s.

Fig. 45. Preliminary Spillway Location. Excavation Modification No. 2, with Piers. Recommended deflector located at vertical bend. Q = 11,500 c.f.s.

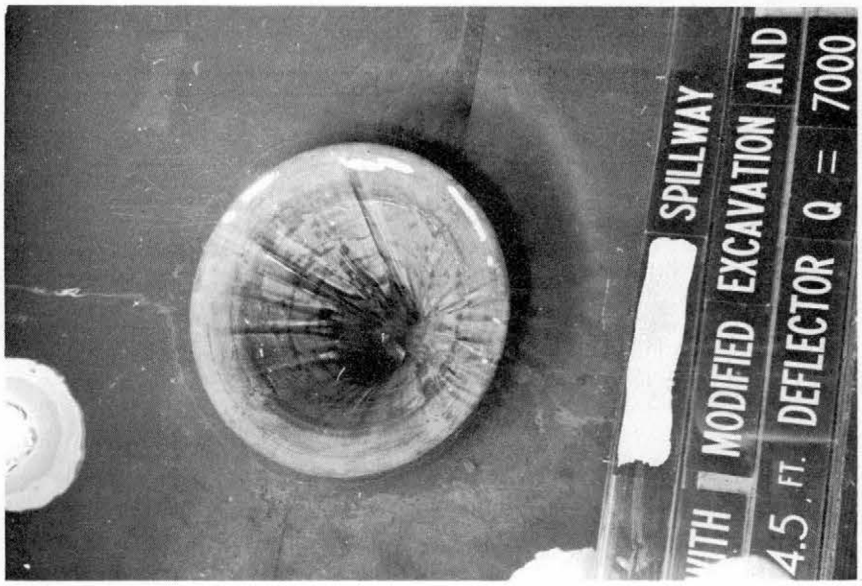


Fig. 46. Preliminary Spillway Location. Excavation Modification No. 3. Recommended deflector at vertical bend. Q = 7000 c.f.s.

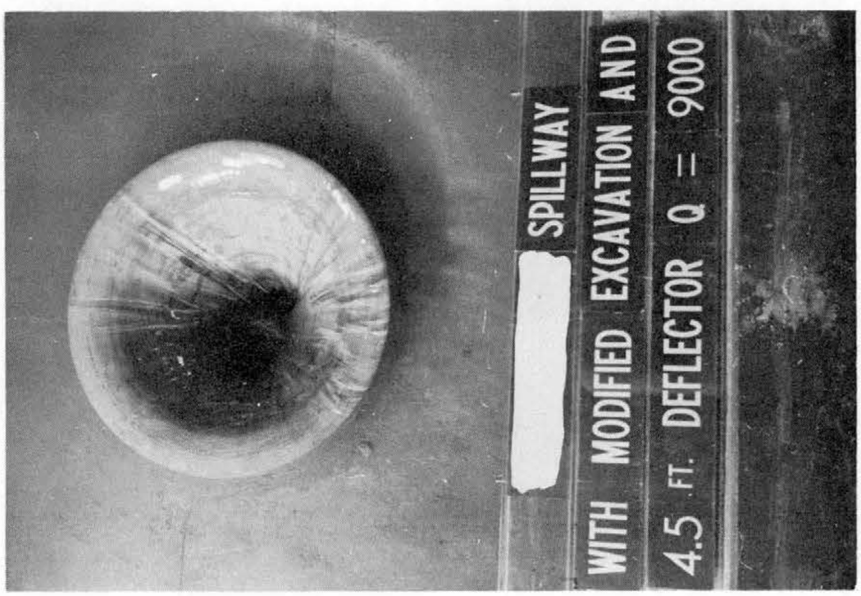


Fig. 47. Preliminary Spillway Location. Excavation Modification No. 3. Recommended deflector at vertical bend. Q = 9000 c.f.s.

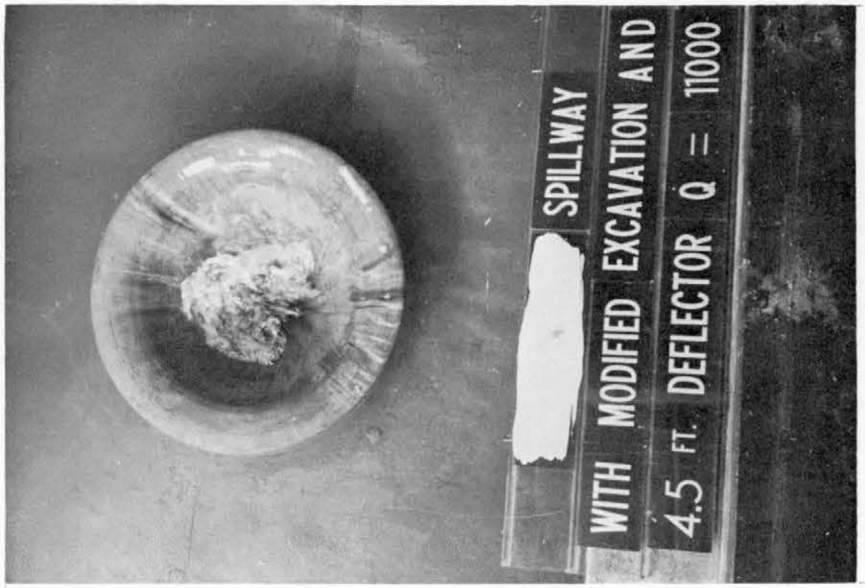


Fig. 48. Preliminary Spillway Location. Excavation Modification No. 3. Recommended deflector at vertical bend. Q = 11,000 c.f.s.

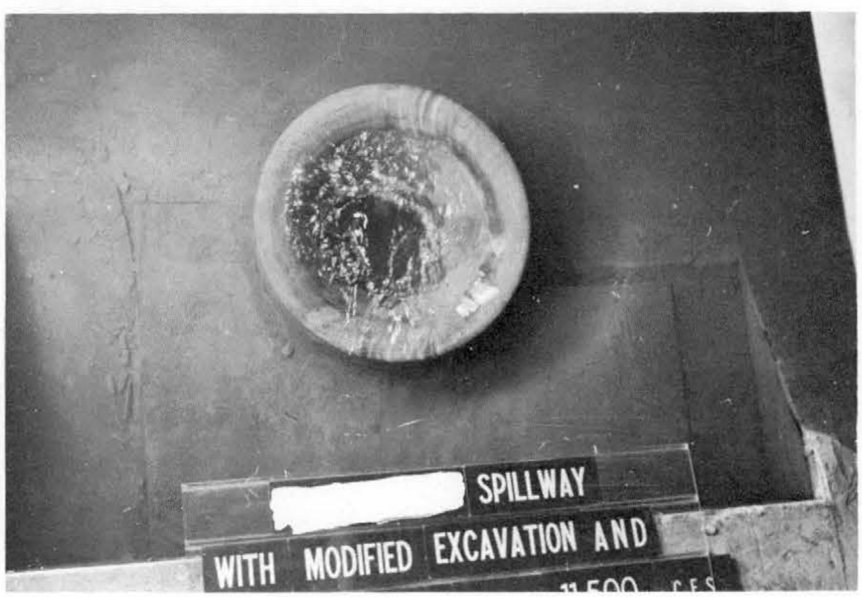


Fig. 49. Preliminary Spillway Location. Excavation Modification No. 3. Recommended deflector at vertical bend. Q = 11,500 c.f.s.

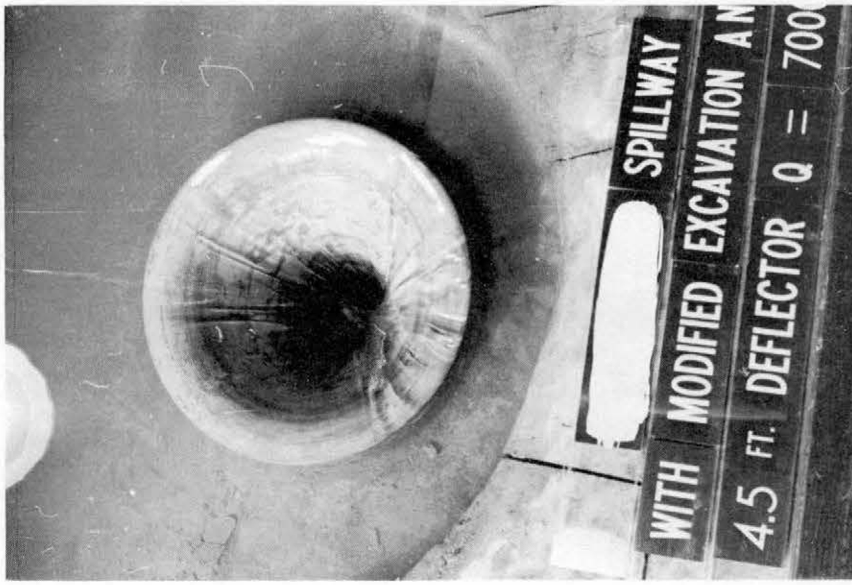
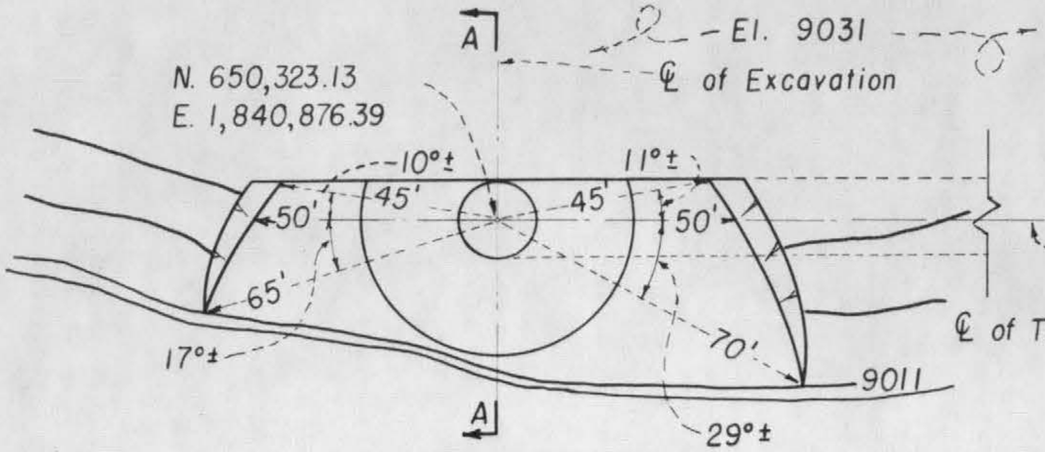


Fig. 50. Preliminary Spillway Location. Excavation Modification No. 4. Recommended deflector located at vertical bend. Q = 7000 c.f.s.

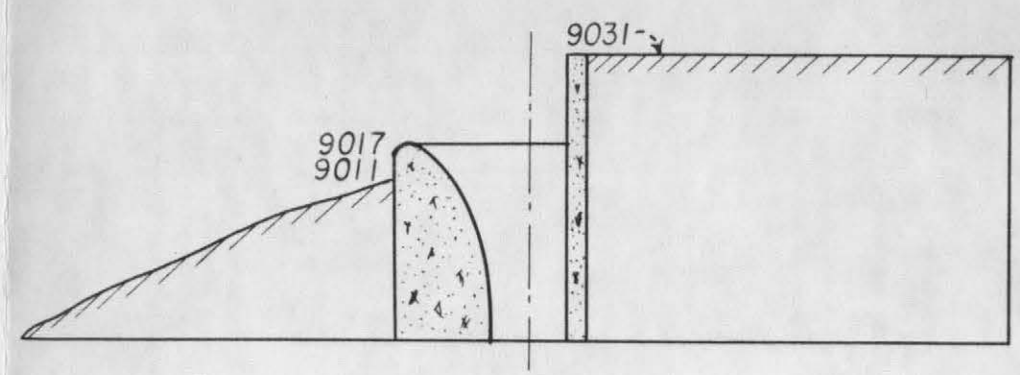
Fig. 51. Preliminary Spillway Location. Excavation Modification No. 4. Recommended deflector located at vertical bend. Q = 9000 c.f.s.

Fig. 52. Preliminary Spillway Location. Excavation Modification No. 4. Recommended deflector located at vertical bend. Q = 11,000 c.f.s.

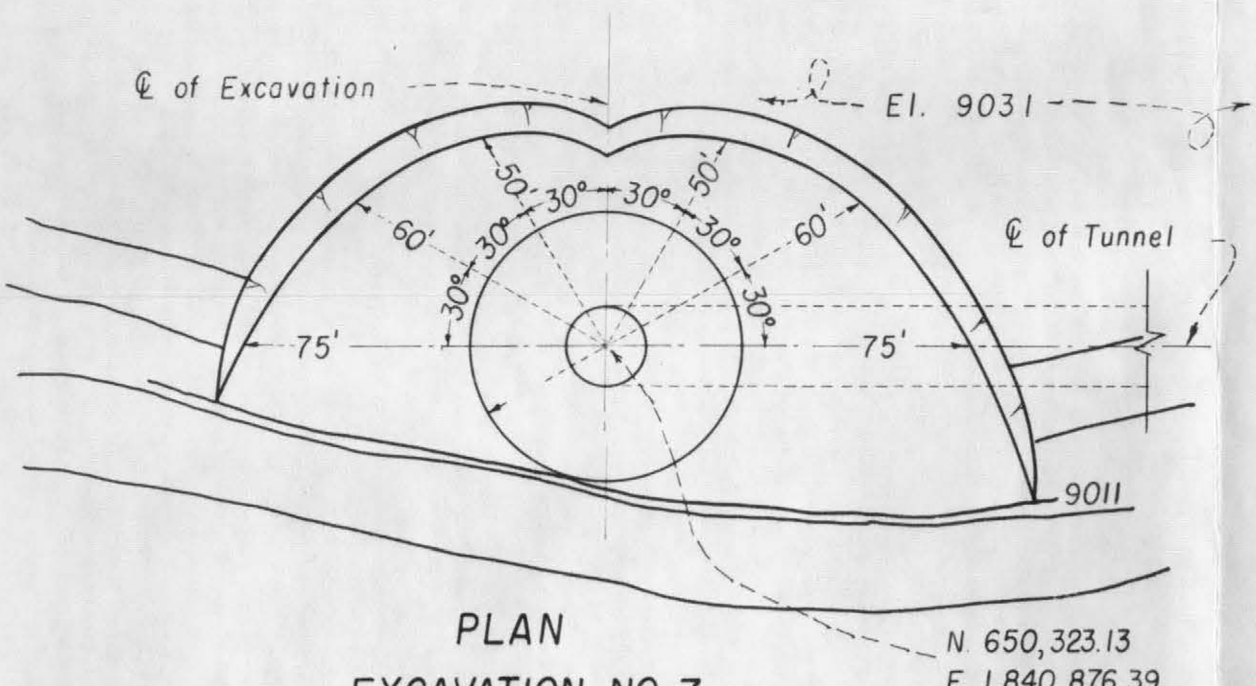
Fig. 53. Preliminary Spillway Location. Excavation Modification No. 4. Recommended deflector located at vertical bend. Q = 11,500 c.f.s.



PLAN
EXCAVATION NO. 6



SECTION A-A



PLAN
EXCAVATION NO. 7

NOTES

See Fig. 15 for preliminary excavation studies.

See Fig. 56 for final spillway location excavation studies.

10 0 50ft.
SCALE

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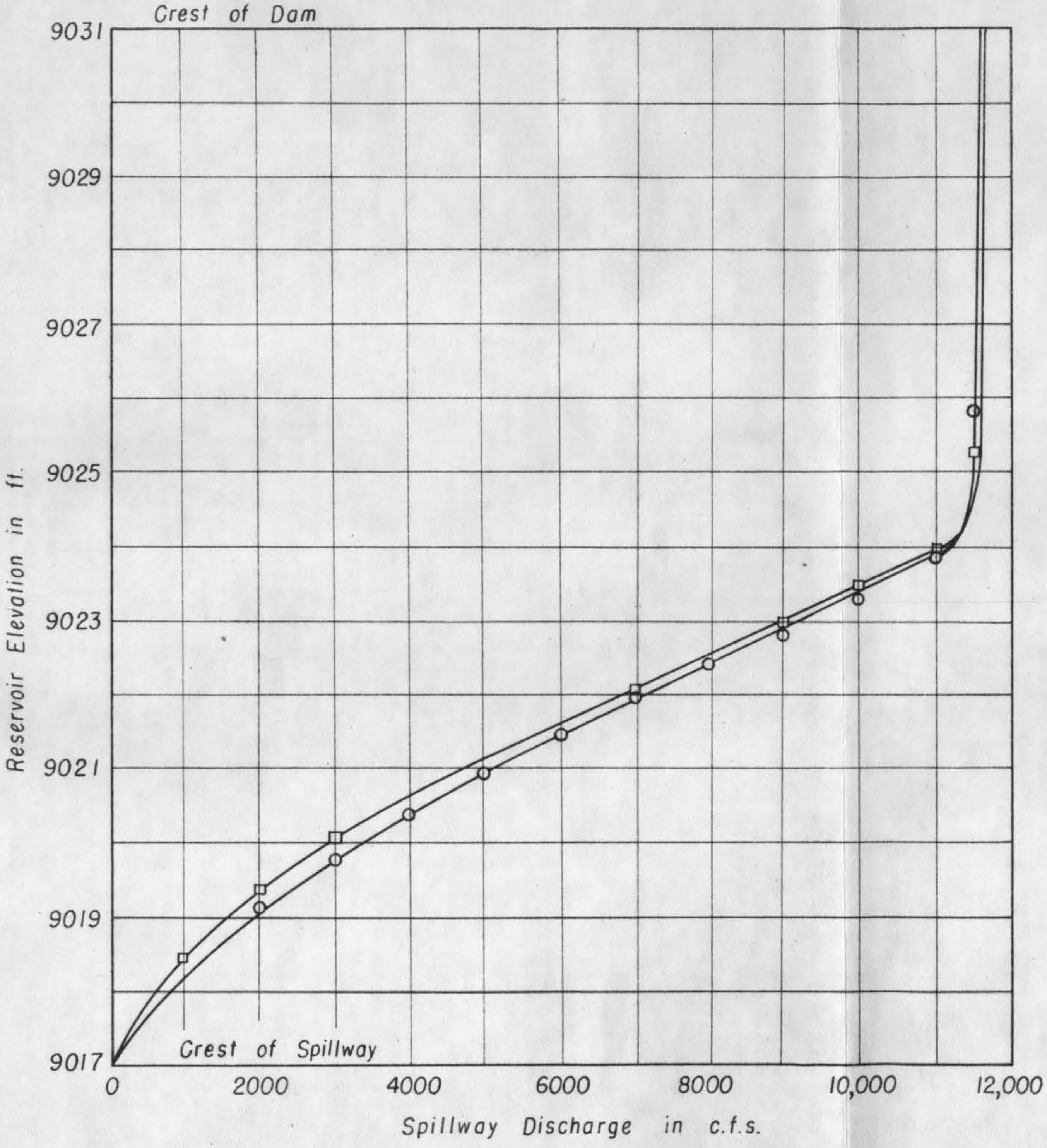
MODEL STUDY
DILLON DAM SPILLWAY
EXCAVATION STUDIES
RELOCATED CREST

Drawn by___R. P. M. Traced by_ A.A.E.I. M.C.C. Checked by___S. K.	Approved S. Karaki

2002	JUNE 1959	FIG. 54

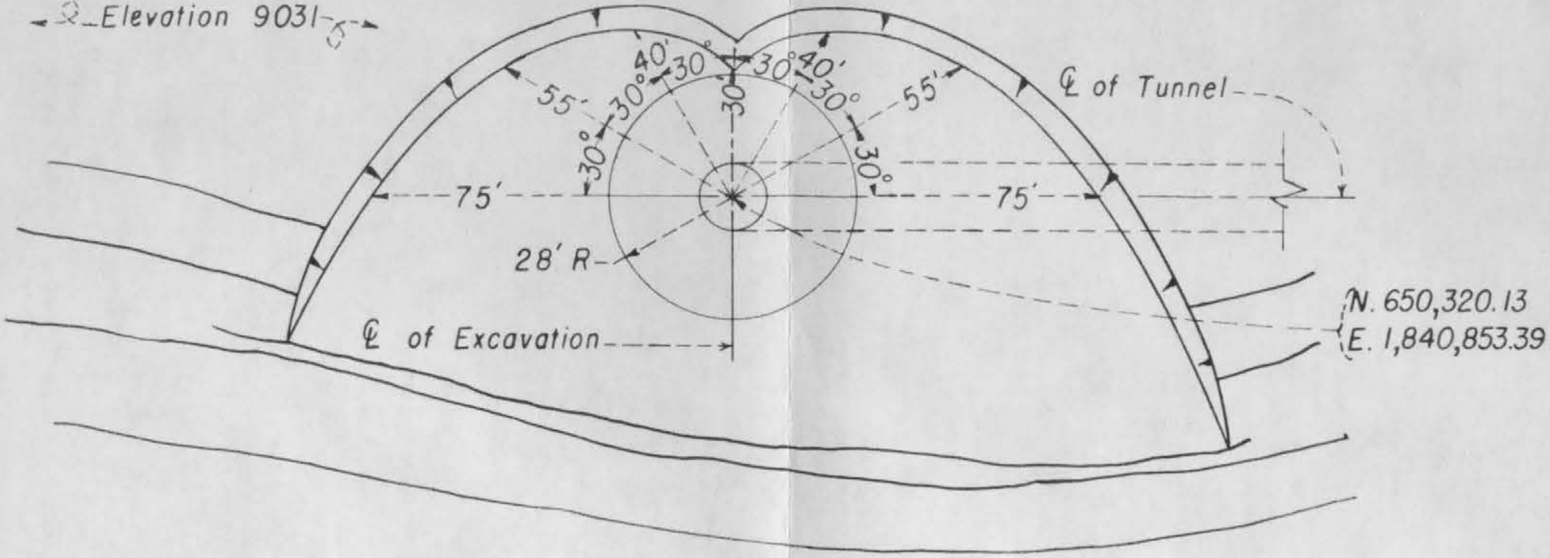


LEGEND

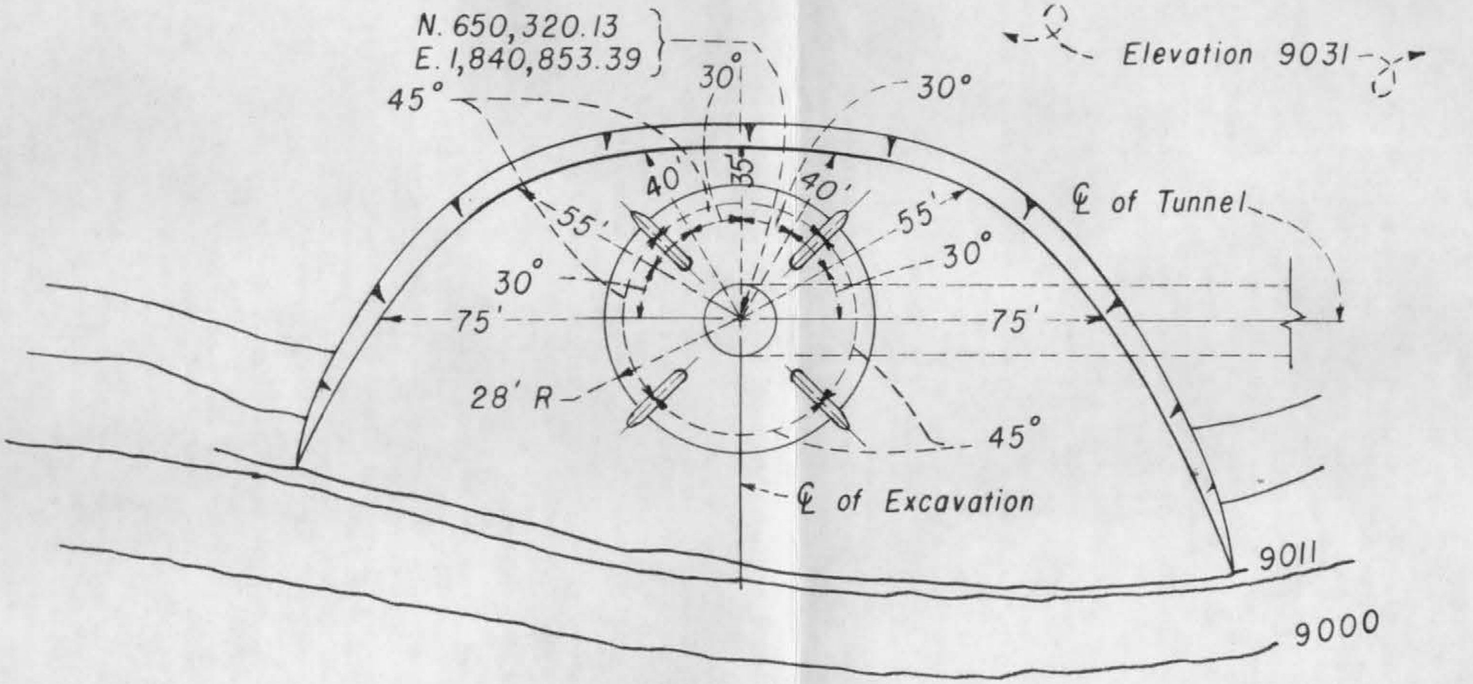
□ Rating curve for relocated crest cut elevation 9011. No piers.

○ Rating curve for relocated crest cut elevation 9000. No piers.

COLORADO STATE UNIVERSITY RESEARCH FOUNDATION MODEL STUDY DILLON DAM SPILLWAY RATING CURVE OF RELOCATED SPILLWAY EXCAVATION MODIFICATION NO. 7		
Drawn by R. P. M. Traced by R.P.M. M.C.C. Checked by S. K.	Approved S. Karaki	
2002	JUNE 1959	FIG. 55



PLAN
EXCAVATION NO. 8



PLAN
EXCAVATION NO. 9
RECOMMENDED

10 0 50 ft.
SCALE

NOTES

See Fig. 98 for design drawing of the shaft spillway.

Crest pier dimensions are shown on Fig. 26.

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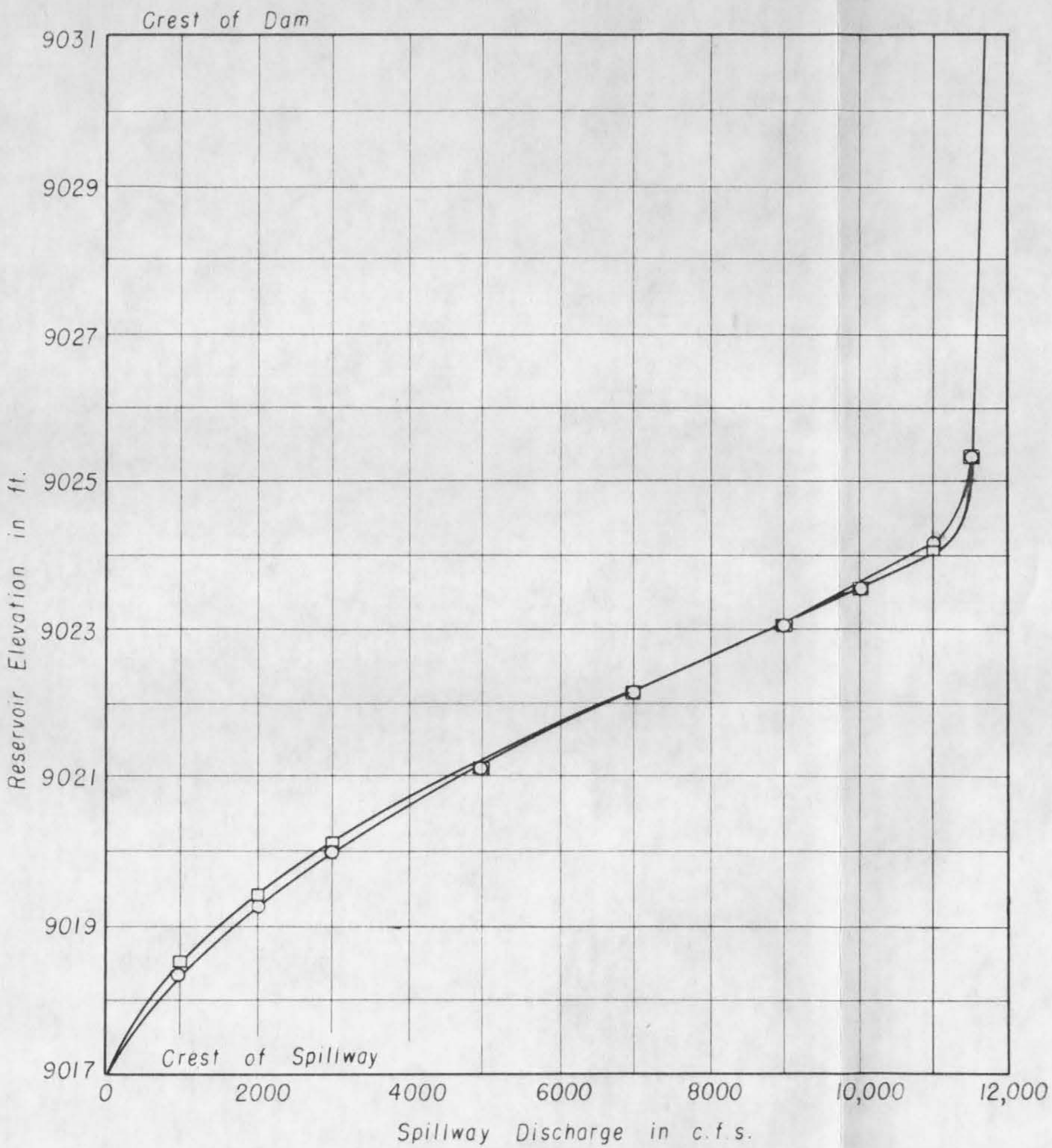
MODEL STUDY
DILLON DAM SPILLWAY

EXCAVATION STUDIES
FINAL SPILLWAY LOCATION

Drawn by R.P.M. Traced by R.P.M. Checked by S.K.	Approved S. Karaki

2002	JUNE 1959	FIG. 56



LEGEND

□ Rating curve for relocated crest cut elevation 9011. No piers.

○ Rating curve for final crest location cut elevation 9011. No piers.

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MODEL STUDY
DILLON DAM SPILLWAY

SPILLWAY RATING CURVE RELOCATED CREST

Drawn by___R. P. M. Traced by_A.A.El. M.C.C. Checked by__ S. K.	Approved S. Karaki __

2002	JUNE 1959	FIG. 57

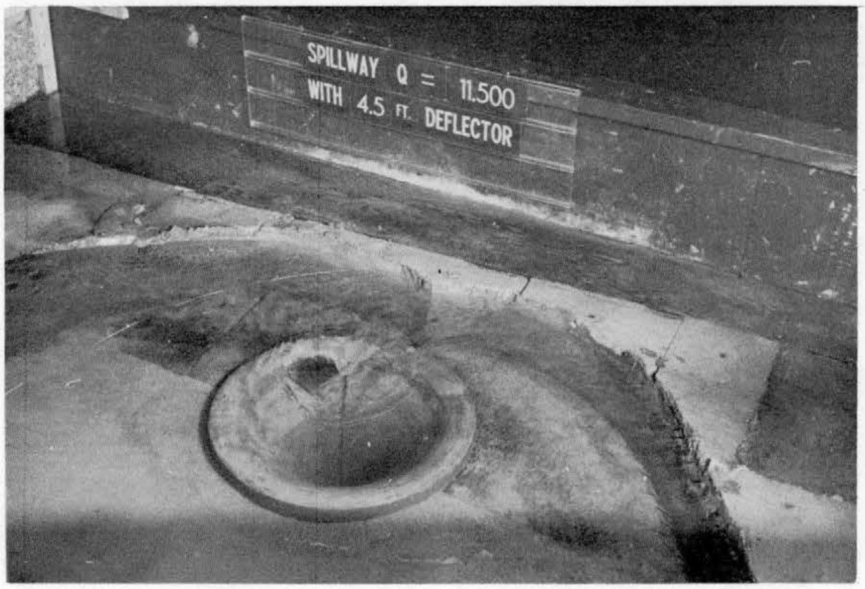


Fig. 58. Final Spillway Location. Vortex at Spillway Crest. Q = 11,500 c.f.s.

Fig. 59. Final Spillway Location. 4 Piers on the Crest. Q = 11,500 c.f.s.

Fig. 60. Final Spillway Location. 6 Piers on the Crest. Q = 11,500 c.f.s.

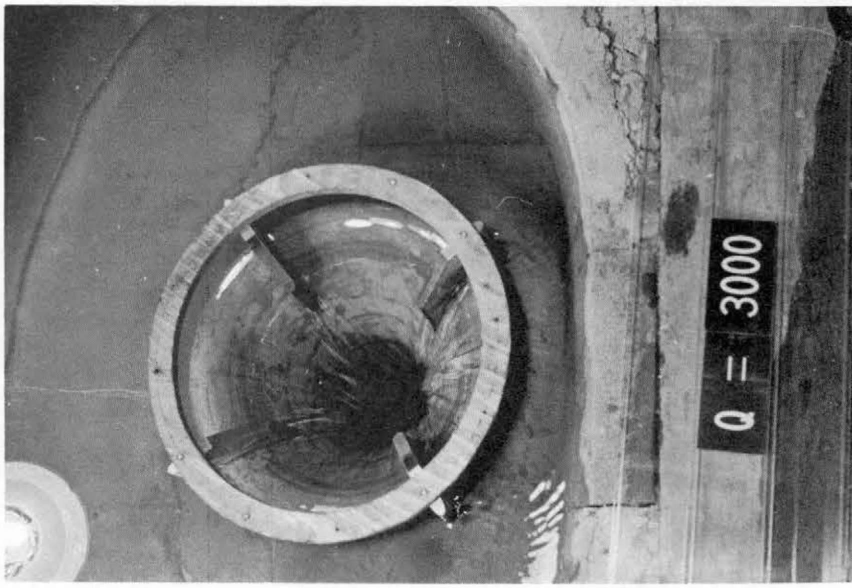


Fig. 61. Final Spillway Location. Recommended Pier Arrangements and Channel Excavation.
Q = 3000 c.f.s.

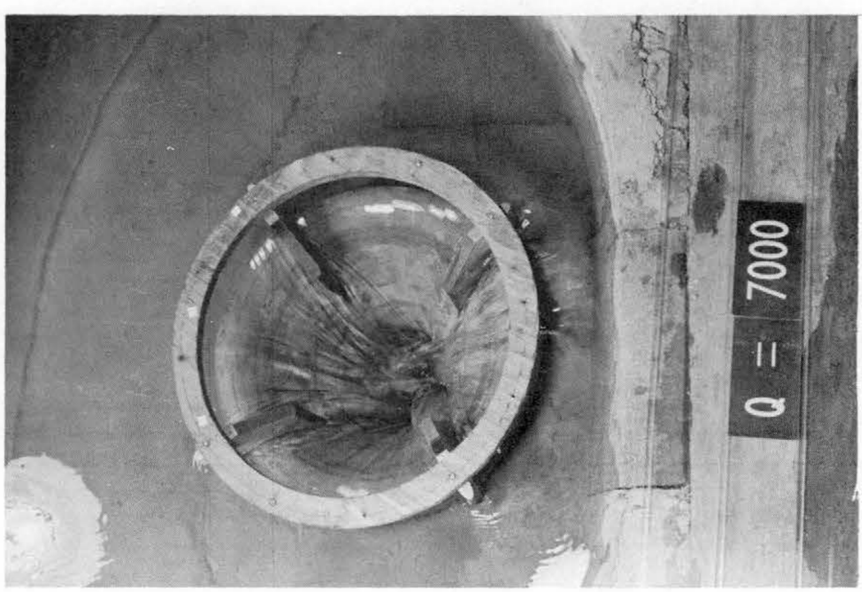


Fig. 62. Final Spillway Location. Recommended Pier Arrangements and Channel Excavation.
Q = 7000 c.f.s.

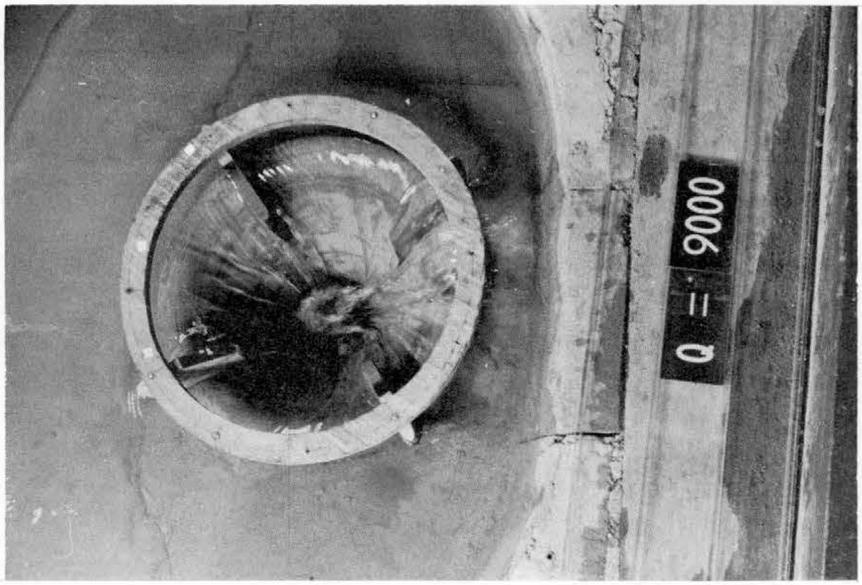
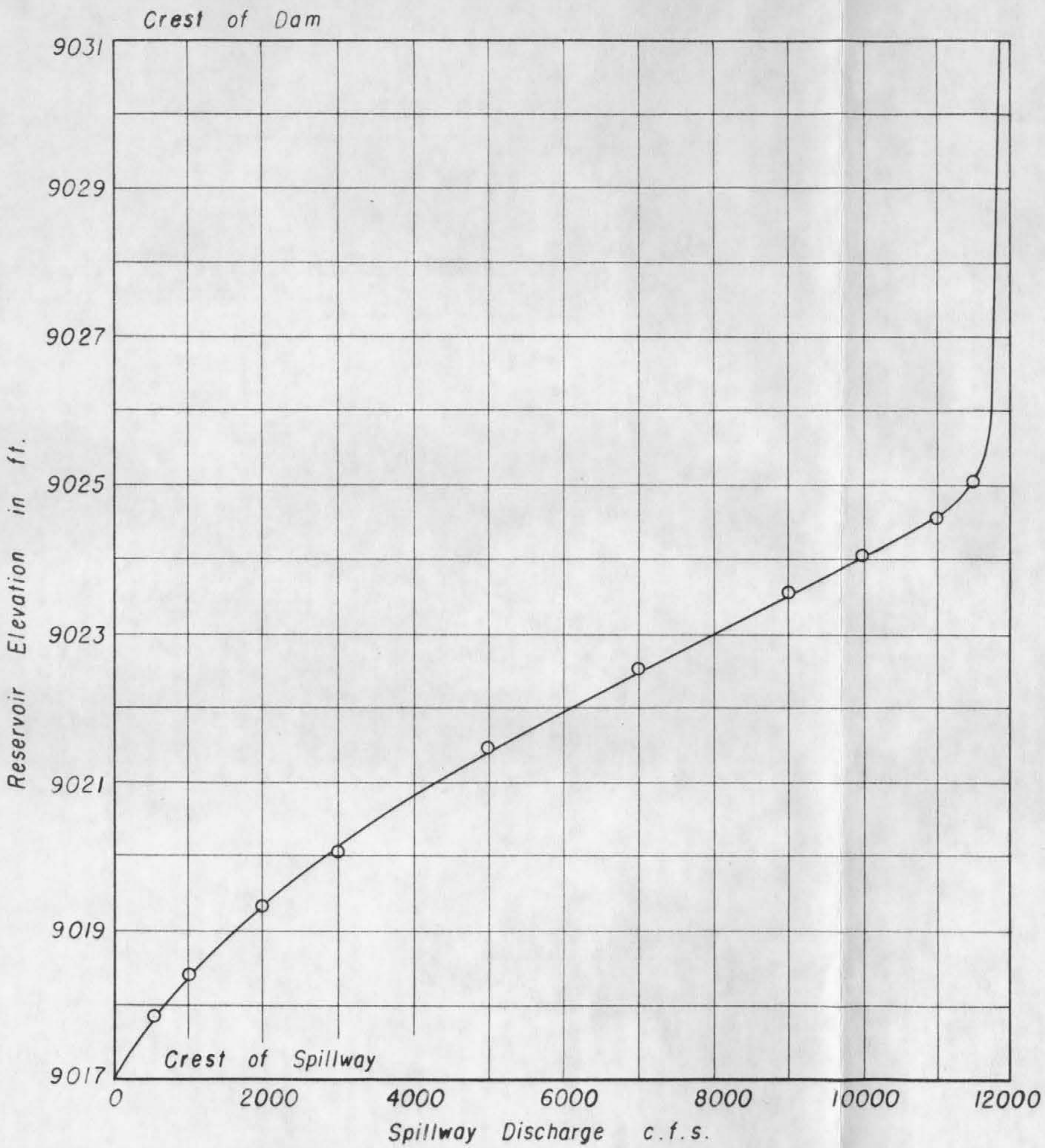


Fig. 63. Final Spillway Location. Recommended Pier Arrangements and Channel Excavation.
$Q = 9000$ c.f.s.

Fig. 64. Final Spillway Location. Recommended Pier Arrangements and Channel Excavation.
$Q = 11,000$ c.f.s.

Fig. 65. Flow Pattern of Recommended Excavation. Q = 7000 c.f.s.

Fig. 66. Flow Pattern of Recommended Excavation. Q = 11,000 c.f.s.



NOTE

See Fig. 56 for Plan of Recommended Spillway for Final Location of the Crest. 4 Piers Were Included.

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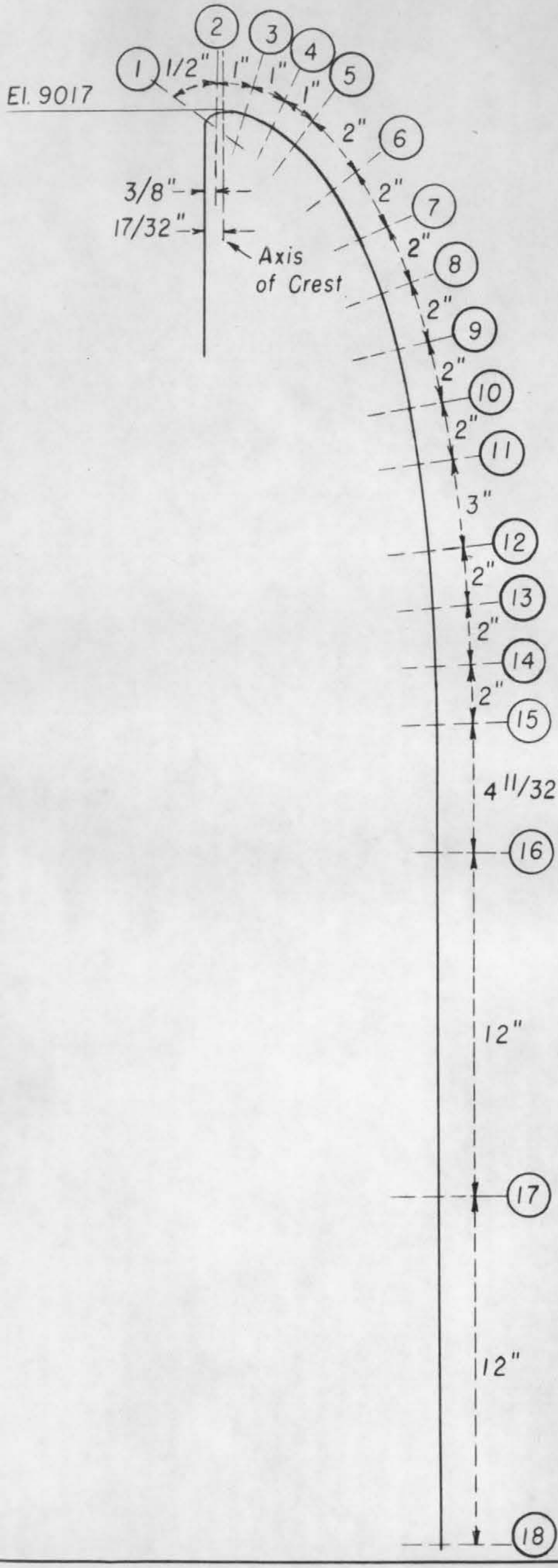
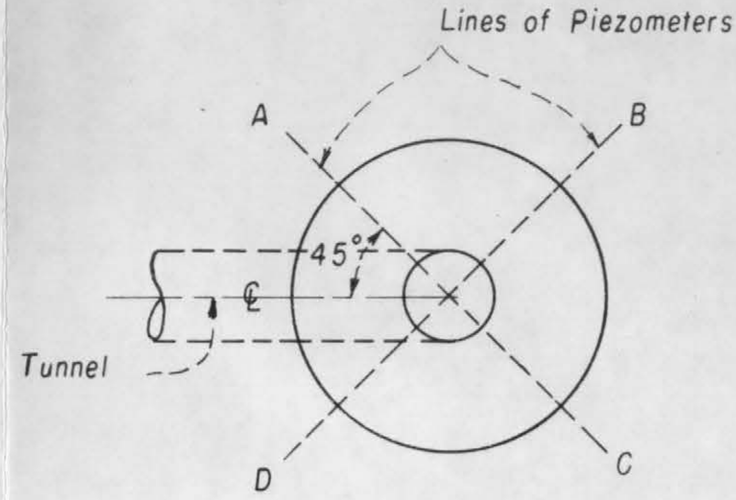
MODEL STUDY
DILLON DAM SPILLWAY

FINAL SPILLWAY RATING CURVE
RECOMMENDED SPILLWAY

Drawn by R. P. M. Traced by R.P.M. M.C.C. Checked by S. K.	Approved S Karaki

2002	JUNE 1959	FIG. 67





NOTE

Numbers in circles denote piezometer numbers. Lines A, B, C, and D, are identical with respect to piezometer location in the profile.

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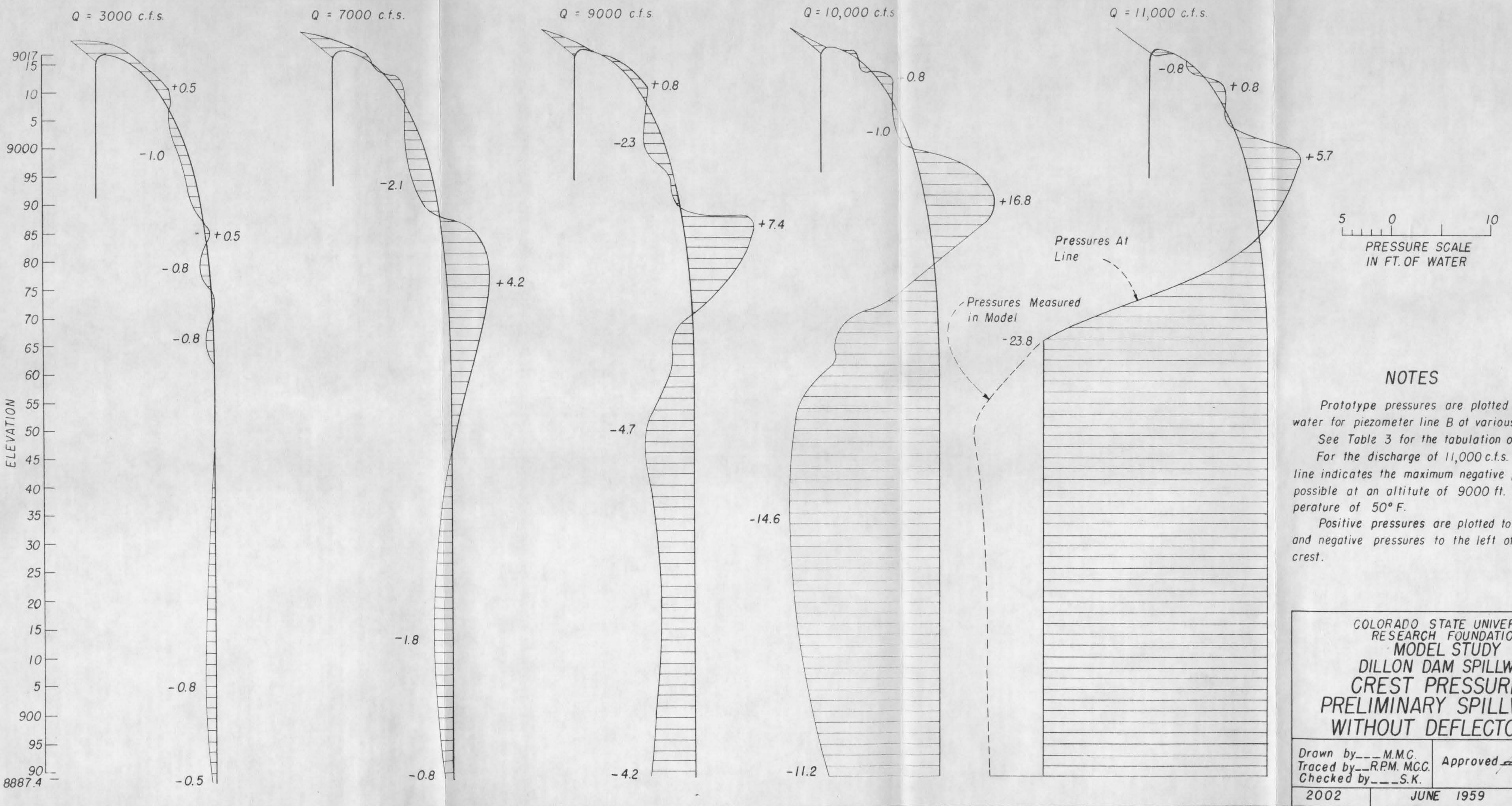
MODEL STUDY
DILLON DAM SPILLWAY

PIEZOMETER LOCATIONS ON SPILLWAY CREST

Drawn by S. S. F. Traced by R.P.M. S.A. Checked by S. K.	Approved S. Karaki

2002	JUNE 1959	FIG. 68



NOTES

Prototype pressures are plotted in feet of water for piezometer line B at various discharges.

See Table 3 for the tabulation of pressures.

For the discharge of 11,000 c.f.s. the vertical line indicates the maximum negative pressure possible at an altitute of 9000 ft. and a temperature of 50° F.

Positive pressures are plotted to the right and negative pressures to the left of the spillway crest.

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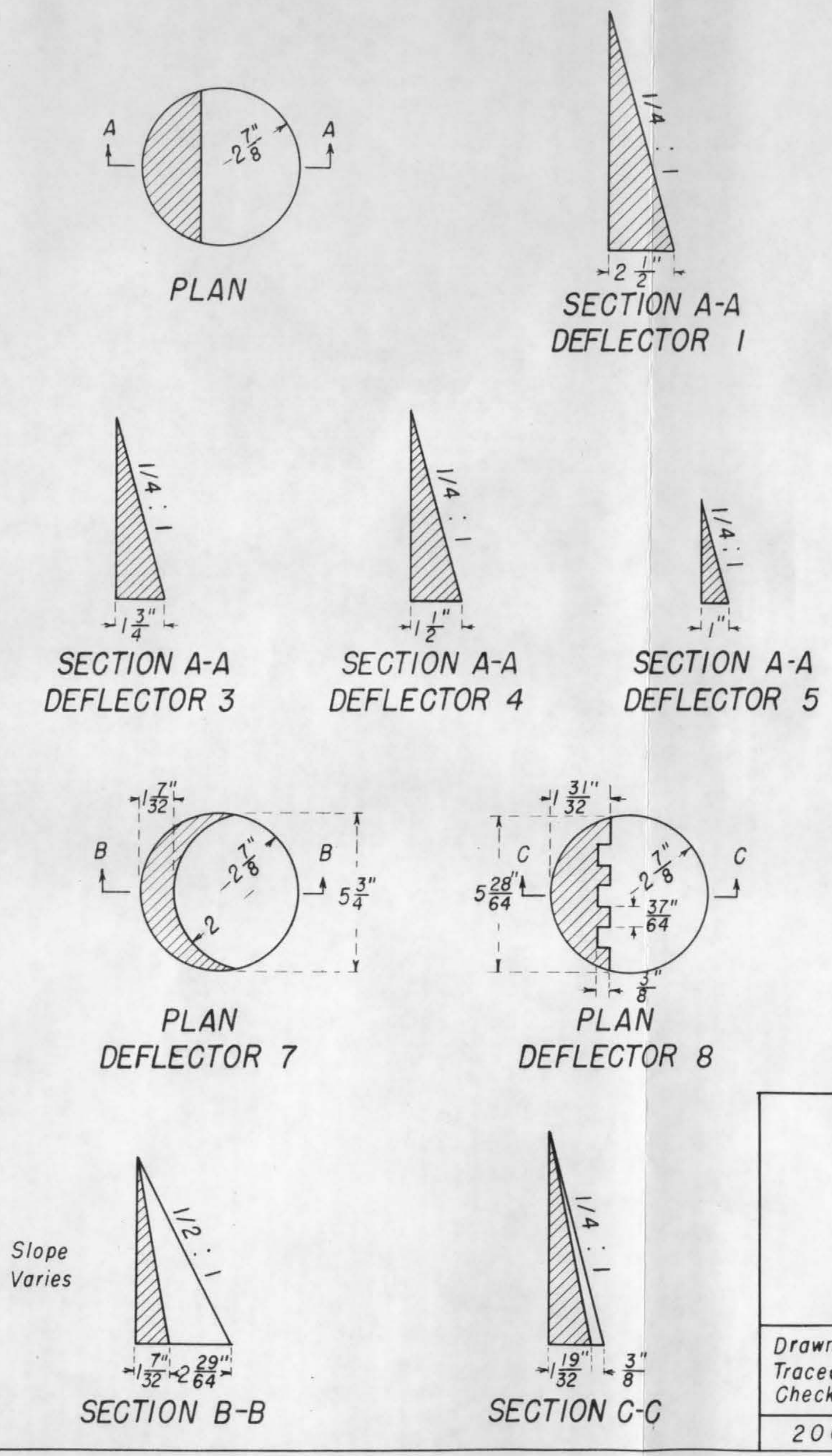
MODEL STUDY
DILLON DAM SPILLWAY
CREST PRESSURES
PRELIMINARY SPILLWAY
WITHOUT DEFLECTOR

Drawn by M.M.C. Traced by R.P.M. M.C.C. Checked by S.K.	Approved S. Karaki
2002	JUNE 1959	FIG. 69



NOTE

Location of deflectors is referenced to the bottom edge. See Figs. 22 and 71 for effects on the Spillway Rating Curve.

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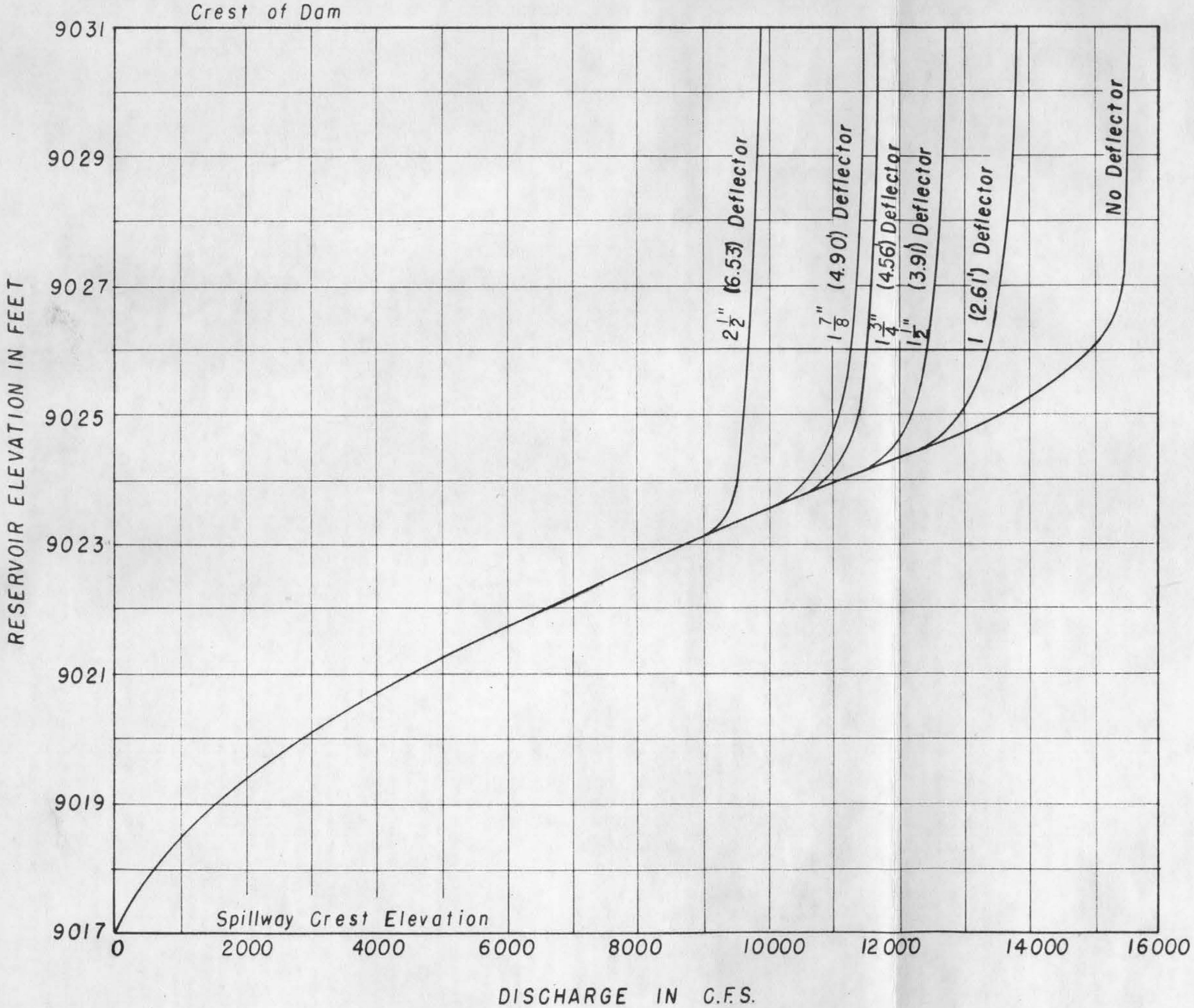
MODEL STUDY
DILLON DAM SPILLWAY

DEFLECTORS

Drawn by___S.S.F. Traced by____H.A. Checked by___S.K.	Approved S. Karaki	
2002	JUNE 1959	FIG. 70



NOTE

Deflectors are located with the bottom edge at the point of curvature. See Fig. 70 for deflector sizes.

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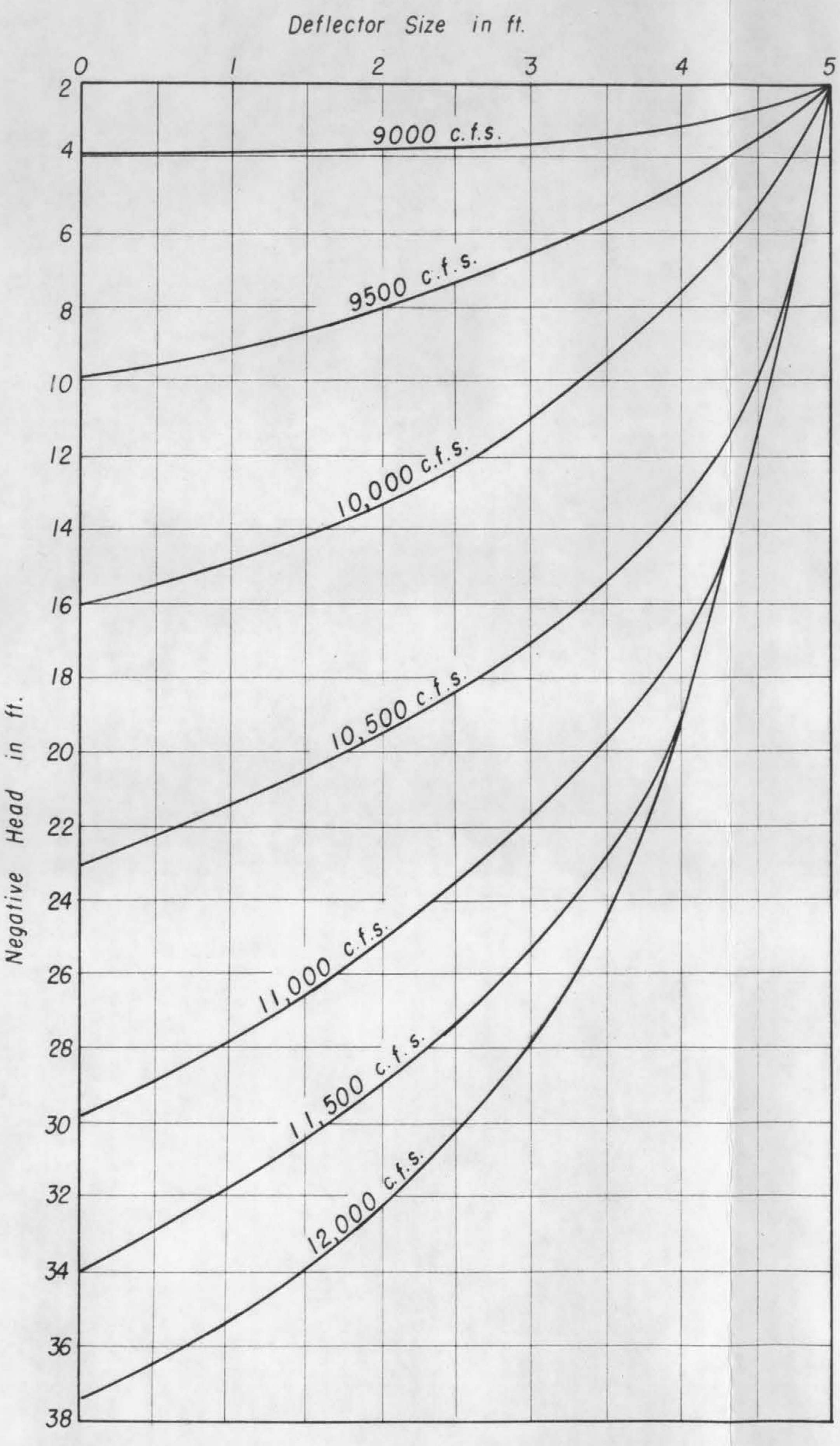
MODEL STUDY
DILLON DAM SPILLWAY

EFFECT OF DEFLECTOR SIZE ON SPILLWAY RATING CURVE

Drawn by S.S.F Traced by A.A.E.I.R.P.M. Checked by S.K.	Approved J. Karaki	
2002	JUNE 1959	FIG. 71



NOTE

Data for this drawing was obtained from Table 4 and 5. Curves were used to determine proper size of deflector.

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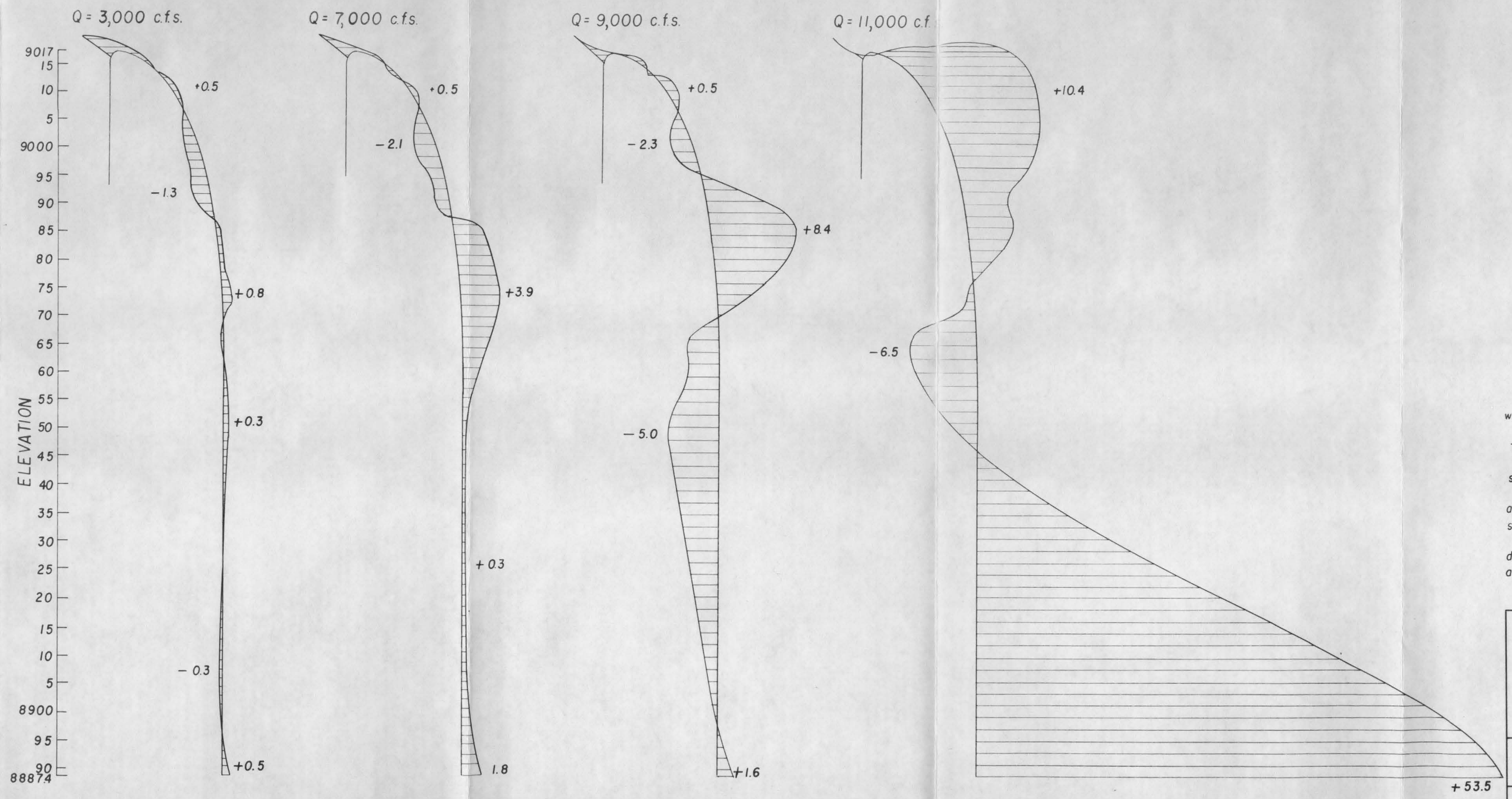
MODEL STUDY
DILLON DAM SPILLWAY

VARIATION OF NEGATIVE HEAD WITH DEFLECTOR SIZE & DISCHARGE

Drawn by S.S.F. Traced by R.P.M. M.C.C. Checked by S.K.	Approved S. Karaki

2002	JUNE 1959	FIG. 72



NOTES

Prototype pressures are plotted in feet of water, for line B at the various discharges.

For complete tabulation of pressures see Table 5.

Pressures are plotted normal to the spillway face.

Positive pressures are shown to the right and negative pressures to the left of the spillway crest.

Pressures were measured with recommended deflector size of 4.5 feet located directly above the p.c. of the vertical bend.

COLORADO STATE UNIVERSITY
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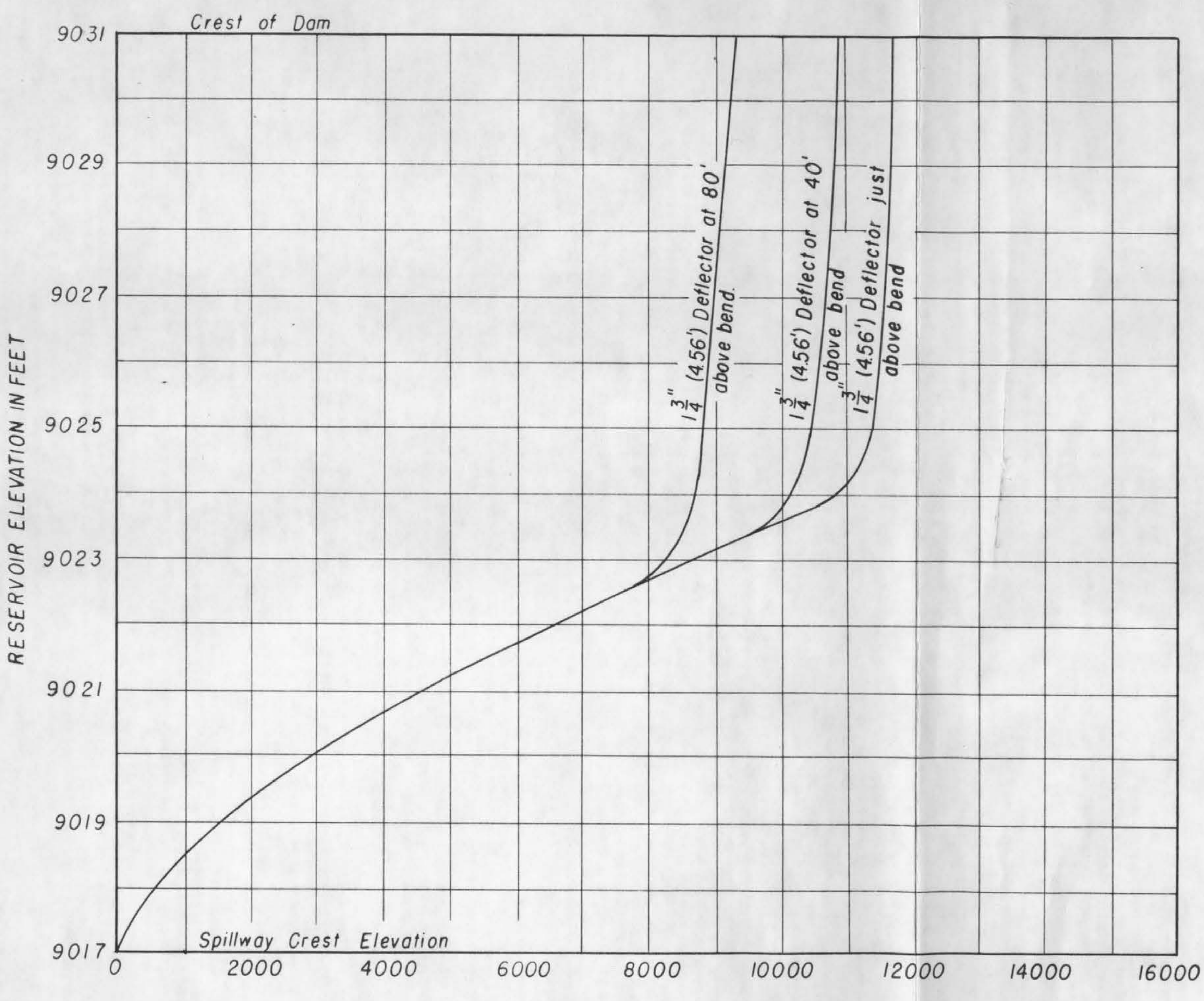
MODEL STUDY
DILLON DAM SPILLWAY

CREST PRESSURES
FINAL SPILLWAY WITH
RECOMMENDED DEFLECTOR

Drawn by__ M.C.C
Traced by__ R.P.M.
Checked by__ S.K.
Approved by S. Karaki

2002 | JUNE 1959 | FIG. 73



NOTE

The location of the deflector has reference to the bottom edge. See Fig. 70 for shape of deflector.

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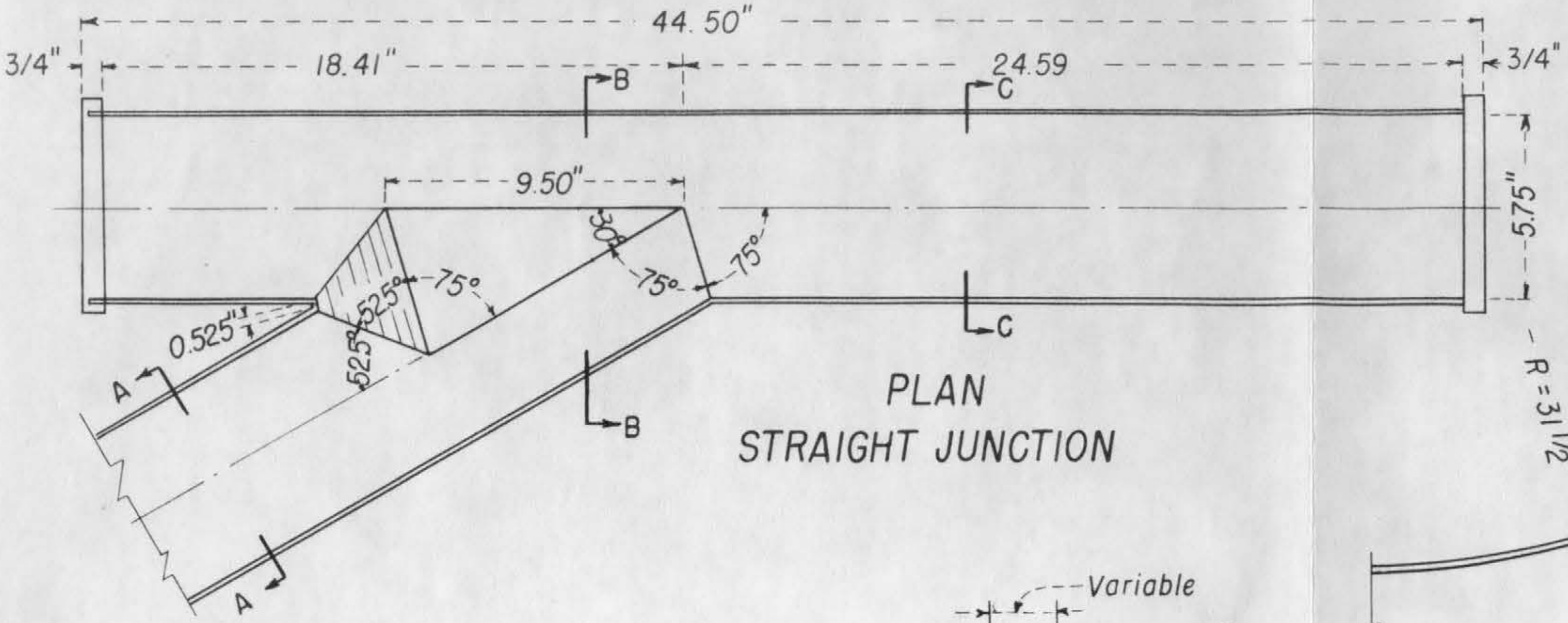
MODEL STUDY
DILLON DAM SPILLWAY

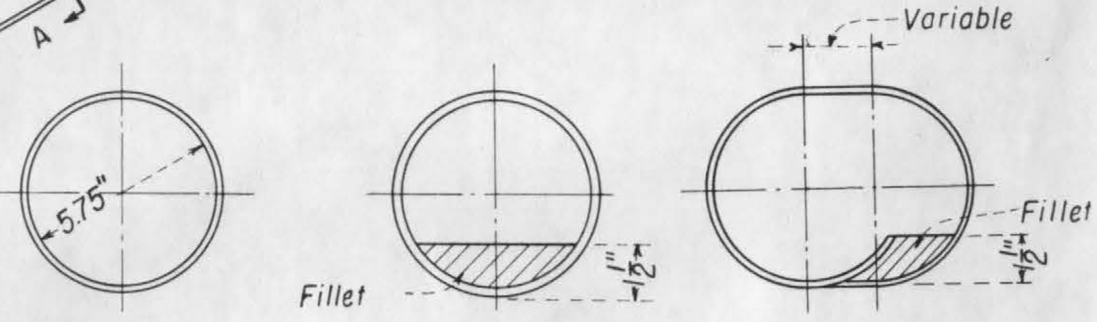
EFFECT OF DEFLECTOR LOCATION ON SPILLWAY RATING CURVE

Drawn by ---- S.S.F. Traced by R.P.M. M.C.C. Checked by --- S.K.	Approved S. Karaki

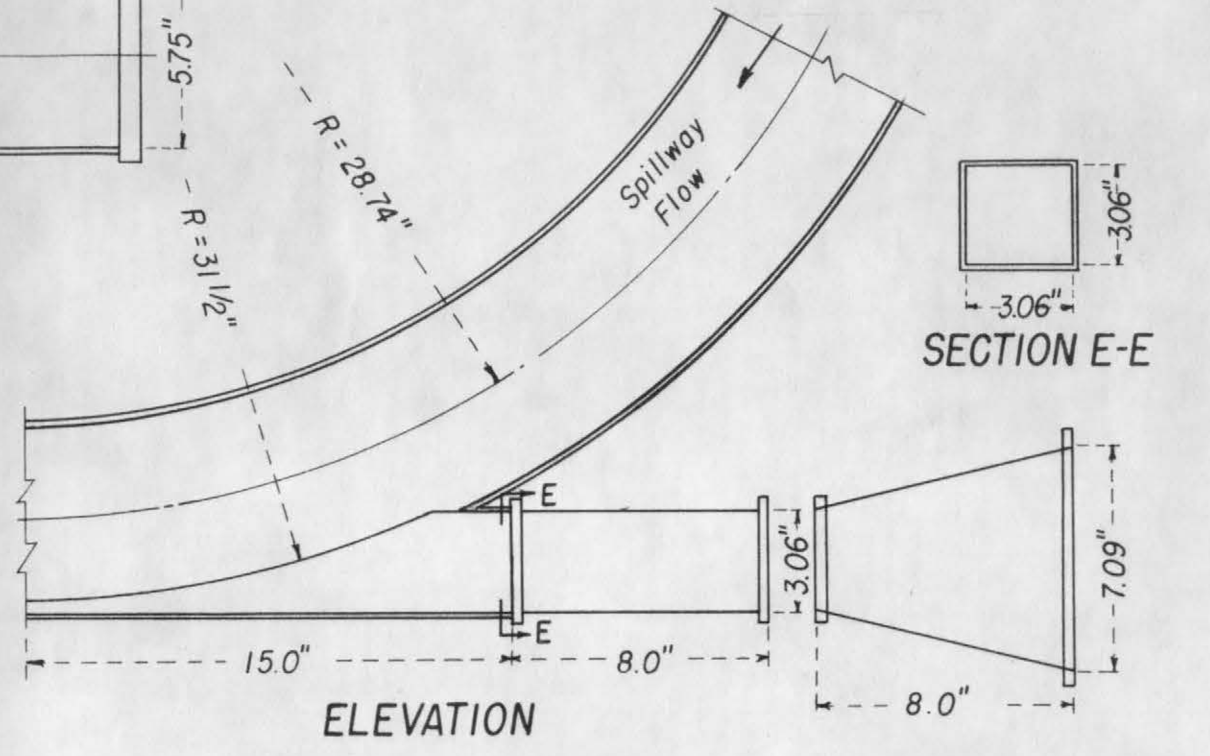
2002	JUNE 1959	FIG. 74



PLAN
STRAIGHT JUNCTION



SECTION C-C SECTION A-A SECTION B-B



SECTION E-E

ELEVATION
RECOMMENDED JUNCTION
BELOW VERTICAL BEND

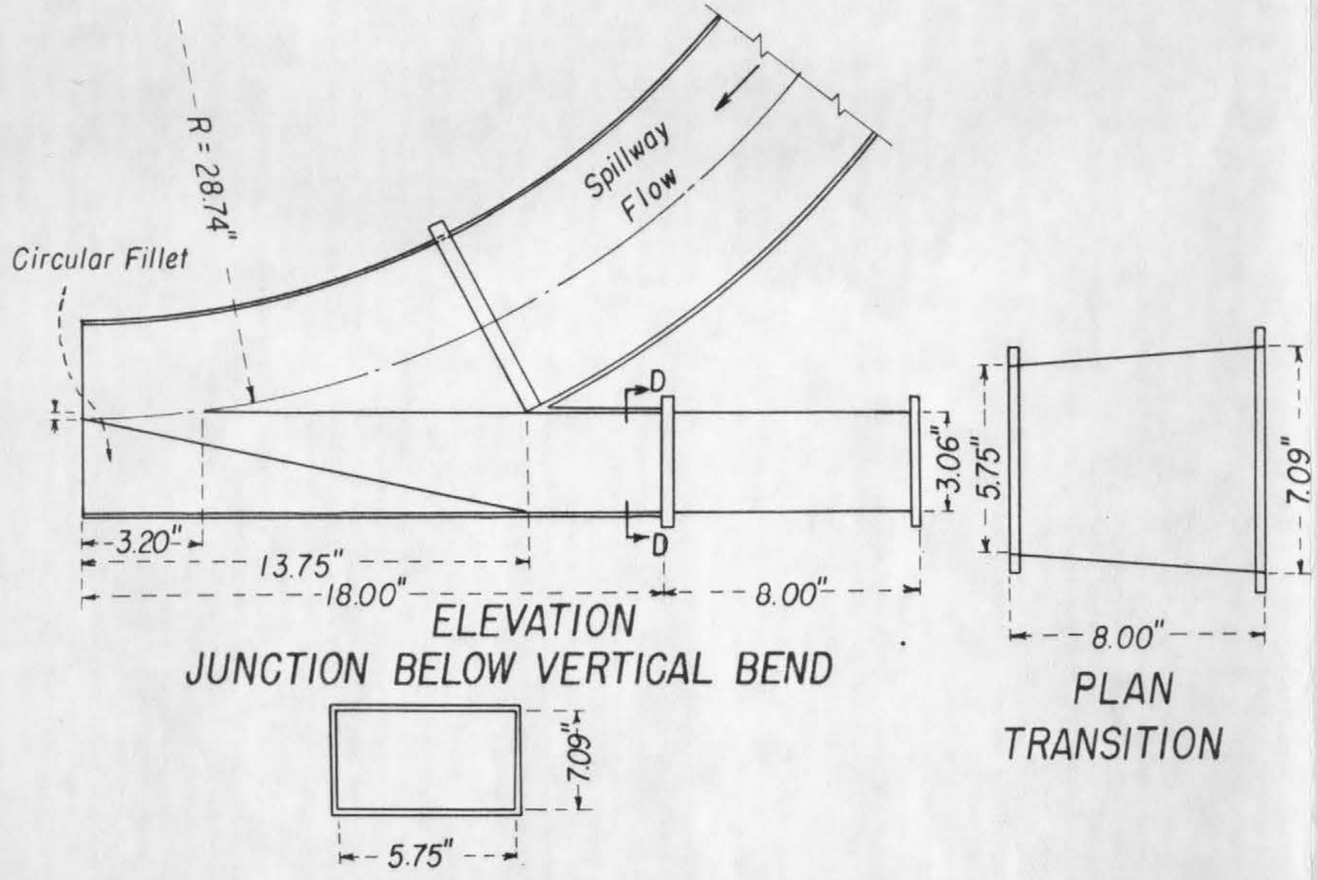
PLAN
TRANSITION

NOTES

The preliminary junction is shown on Fig. 8

The 30° angle straight junction was 25 feet farther downstream than the preliminary junction.

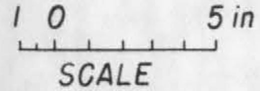
Model dimensions only shown.



ELEVATION
JUNCTION BELOW VERTICAL BEND

PLAN
TRANSITION

SECTION D-D



COLORADO STATE UNIVERSITY
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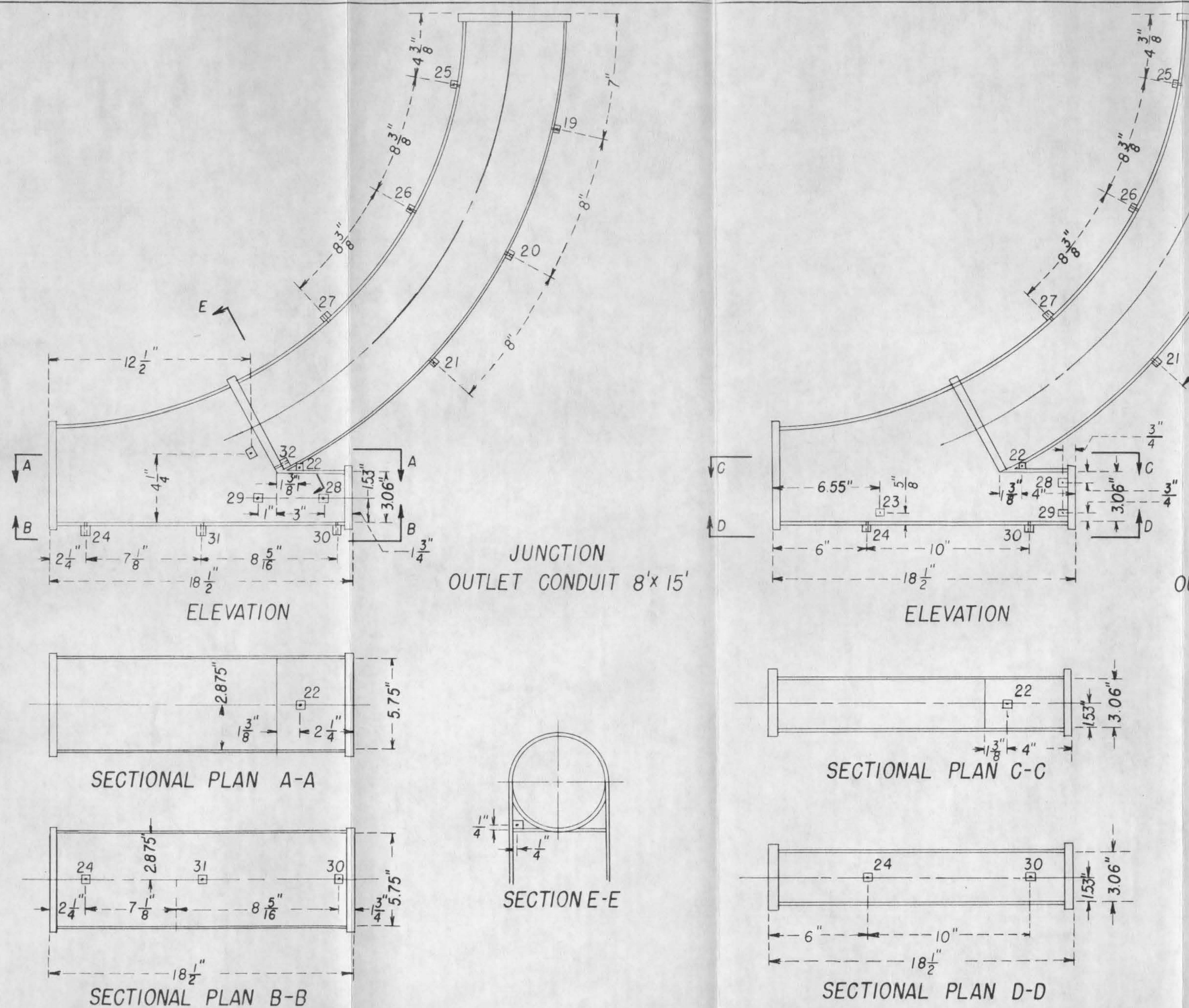
MODEL STUDY
DILLON DAM SPILLWAY

SPILLWAY-OUTLET JUNCTION
PLANS, SECTIONS AND ELEVATIONS

Drawn by ___ M.C.C. Traced by H.A. M.C.C. Checked by __S. K.	Approved S. Karaki
2002	JUNE 1959	FIG. 75a



COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION
MODEL STUDY
DILLON DAM SPILLWAY
SPILLWAY-OUTLET JUNCTIONS
PIEZOMETER LOCATIONS

Drawn by A.A.EI.
Traced by R.P.M. M.C.C.
Checked by S.K.
Approved S. Karaki

2002	JUNE 1959	FIG. 75b

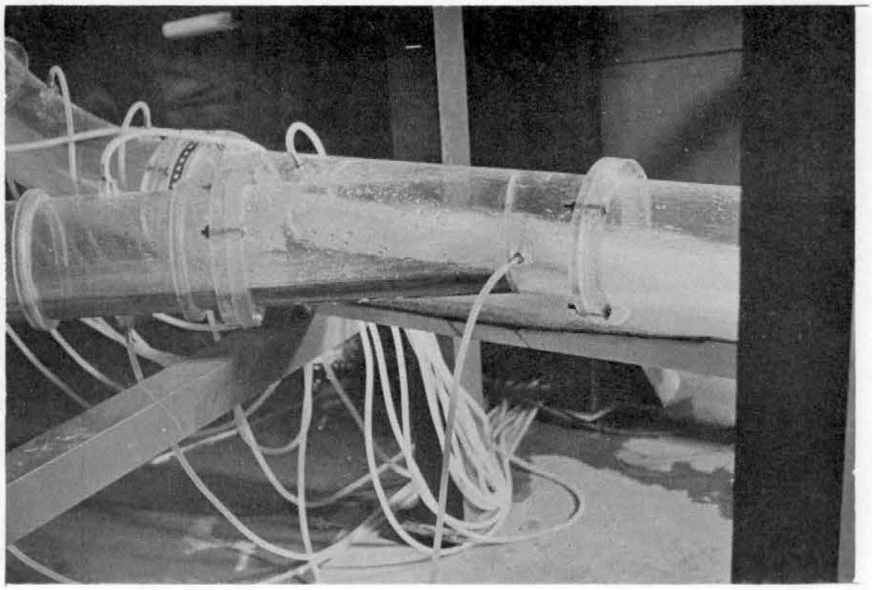


Fig. 76. Straight Outlet Tunnel Junction.
Spillway Q = 5000 c.f.s.
Outlet Q = 0.

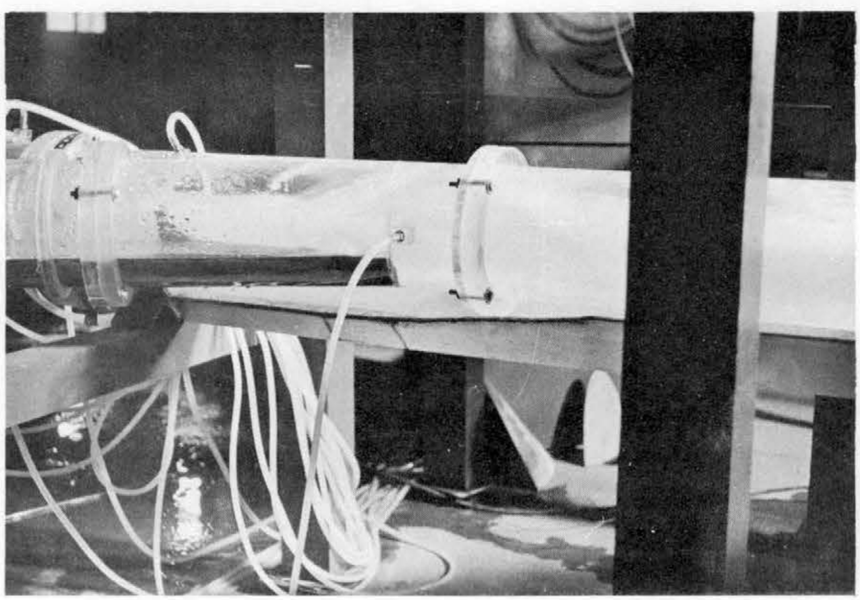


Fig. 77. Straight Outlet Tunnel Junction.
Spillway Q = 9000 c.f.s.
Outlet Q = 0.

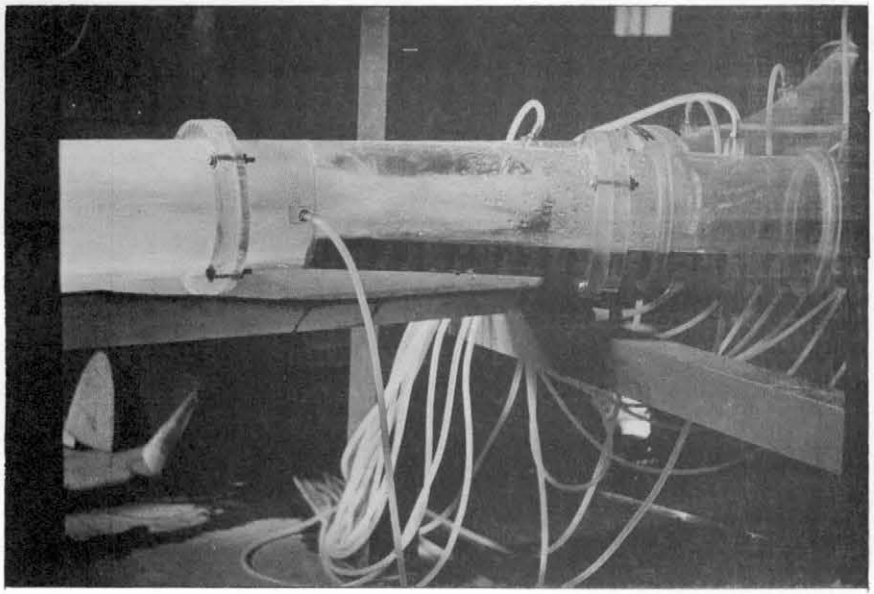


Fig. 78. Straight Outlet Tunnel Junction.
Spillway Q = 11,500 c.f.s.
Outlet Q = 0.

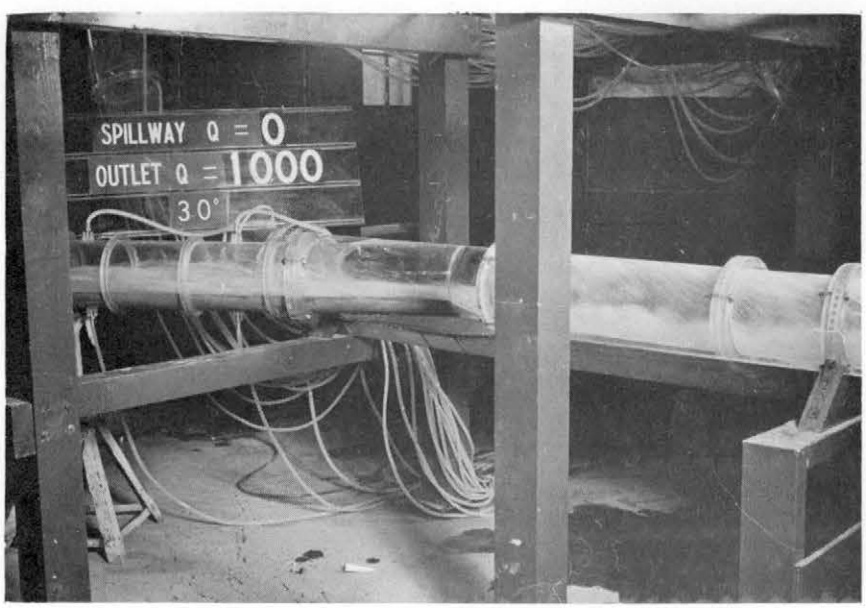


Fig. 79. Straight Outlet Tunnel Junction.
Spillway Q = 0.
Outlet Q = 1000 c.f.s.

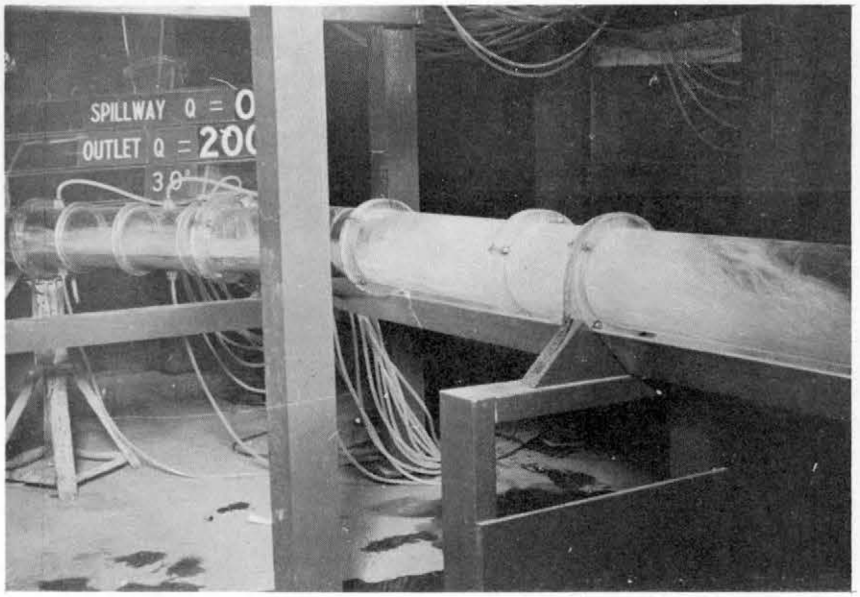


Fig. 80. Straight Outlet Tunnel Junction.
Spillway $Q = 0$.
Outlet $Q = 2000$ c.f.s.

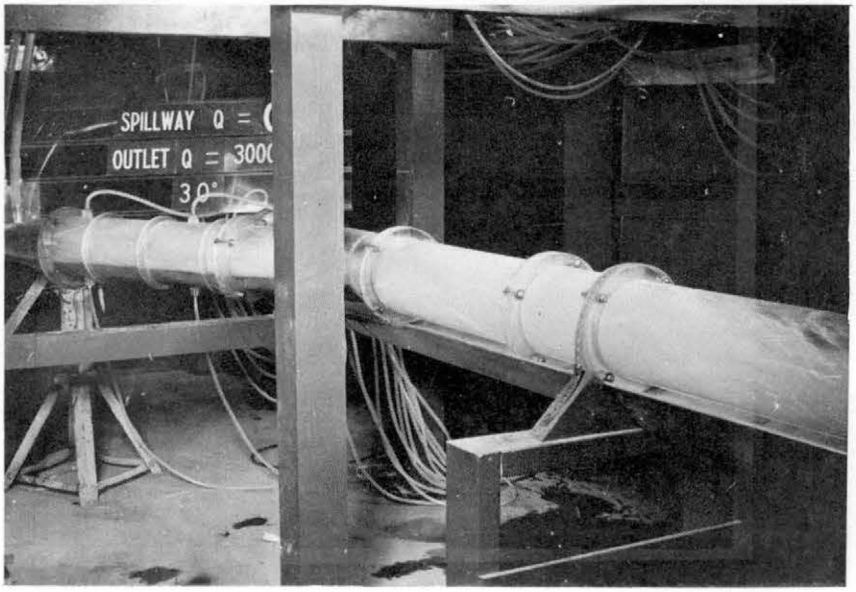


Fig. 81. Straight Outlet Tunnel Junction
Spillway $Q = 0$.
Outlet $Q = 3000$ c.f.s.

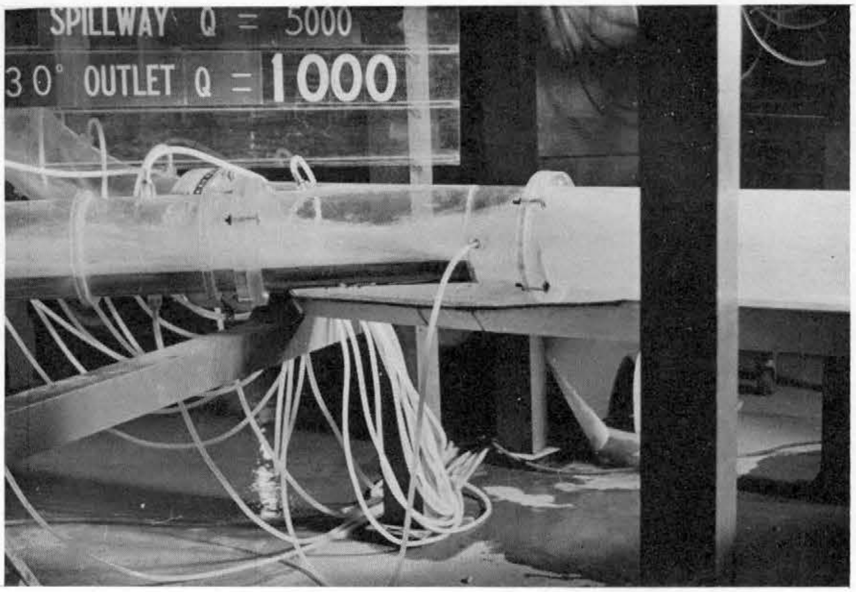


Fig. 82. Straight Outlet Tunnel Junction.
Spillway Q = 5000 c.f.s.
Outlet Q = 1000 c.f.s.

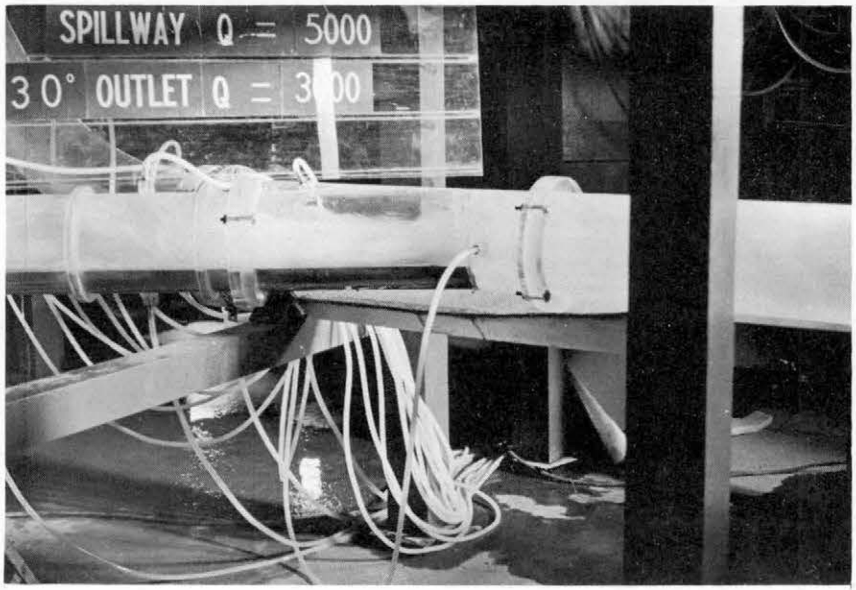


Fig. 83. Straight Outlet Tunnel Junction.
Spillway Q = 5000 c.f.s.
Outlet Q = 3000 c.f.s.

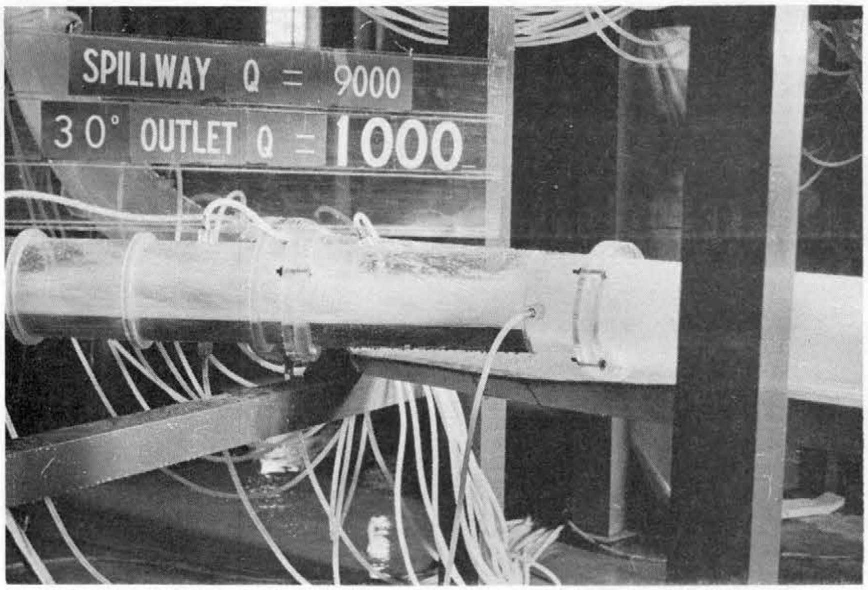


Fig. 84. Straight Outlet Tunnel Junction.
Spillway Q = 9000 c.f.s.
Outlet Q = 1000 c.f.s.

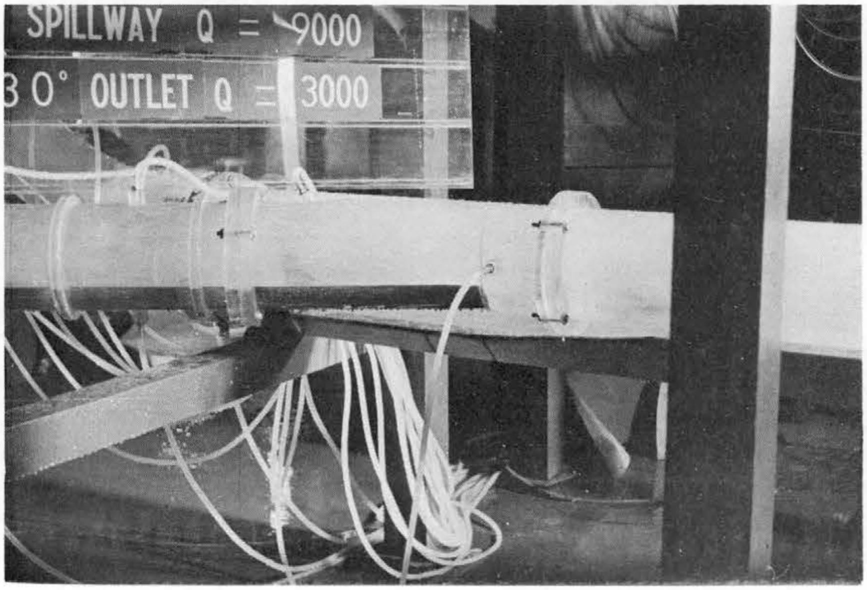


Fig. 85. Straight Outlet Tunnel Junction.
Spillway Q = 9000 c.f.s.
Outlet Q = 3000 c.f.s.

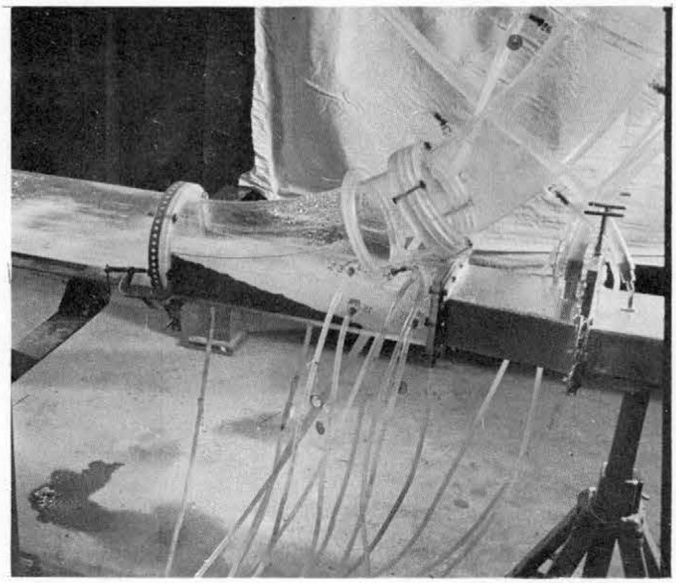


Fig. 86. Junction at Vertical Bend.
Outlet Conduit 8 x 15 ft.
Spillway Q = 0. Outlet Q = 1000 c.f.s.

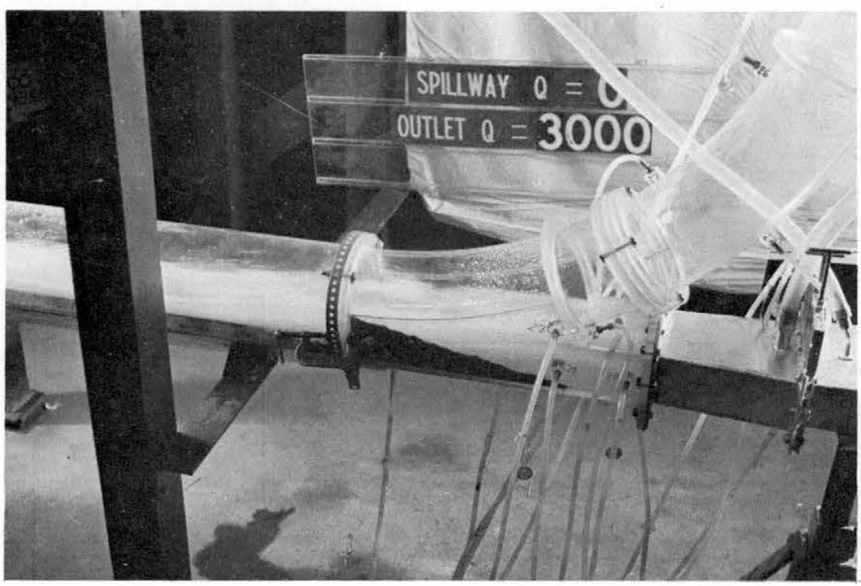


Fig. 87. Junction at Vertical Bend.
Outlet Conduit 8 x 15 ft.
Spillway Q = 0. Outlet Q = 3000 c.f.s.

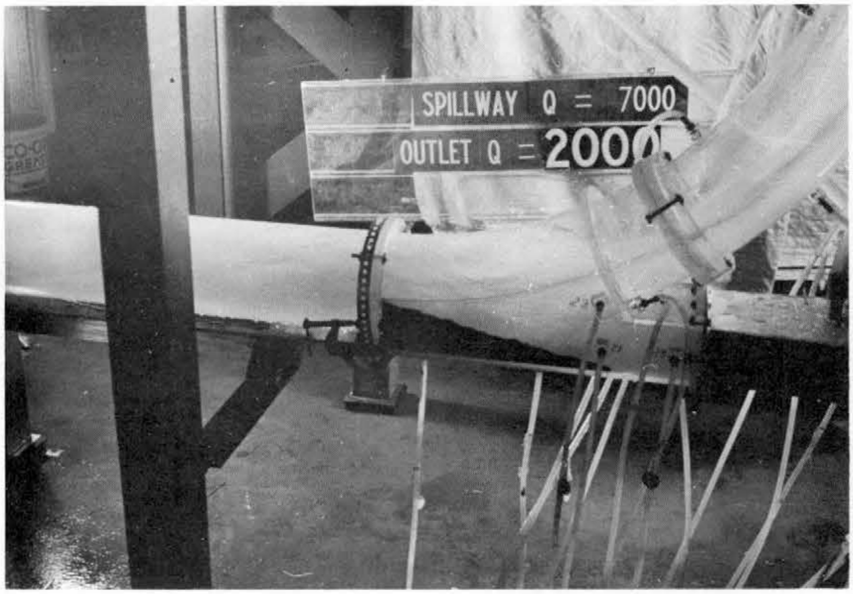


Fig. 88 Junction at Vertical Bend.
Outlet Conduit 8 x 15 ft.
Spillway Q = 7000 c.f.s. Outlet Q = 2000 c.f.s.

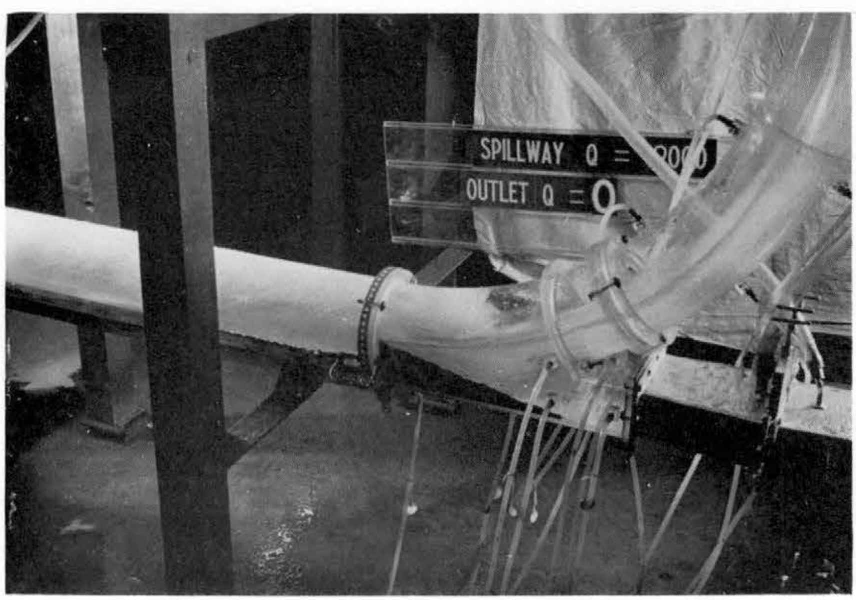


Fig. 89. Junction at Vertical Bend. Outlet Conduit 8 x 15 ft.
Spillway Q = 9000 c.f.s. Outlet Q = 0.

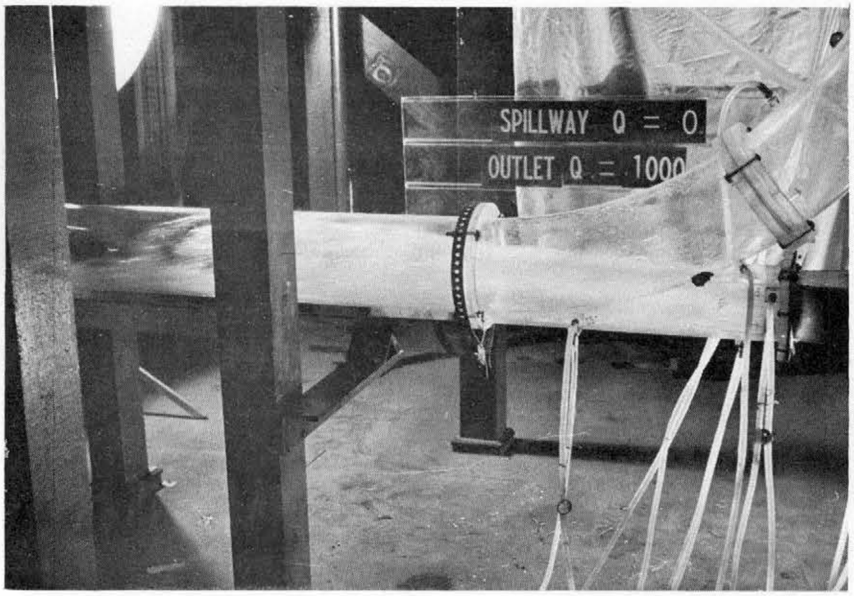


Fig. 90. Outlet Conduit 8 x 8 ft.
Spillway Q = 0. Outlet Q = 1000 c.f.s.

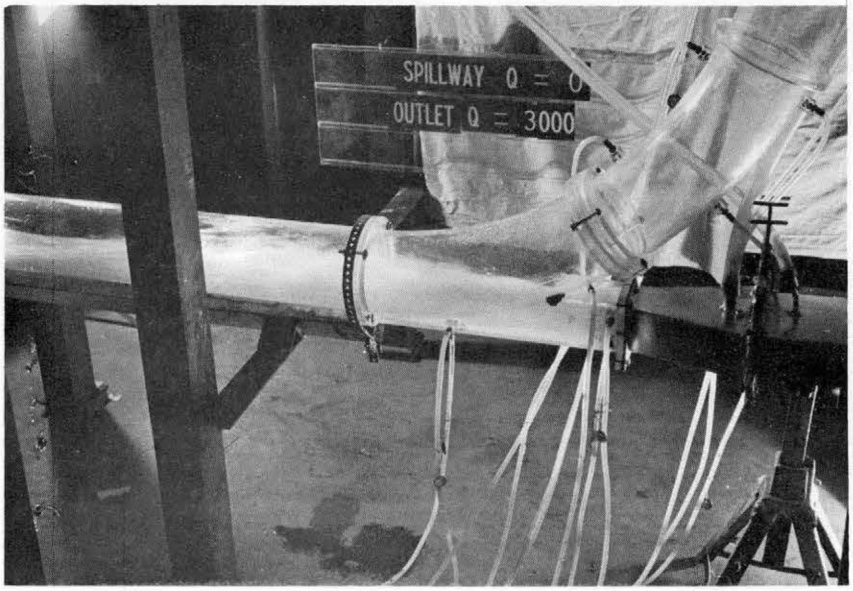


Fig. 91. Outlet Conduit 8 x 8 ft.
Spillway Q = 0. Outlet Q = 3000 c.f.s.

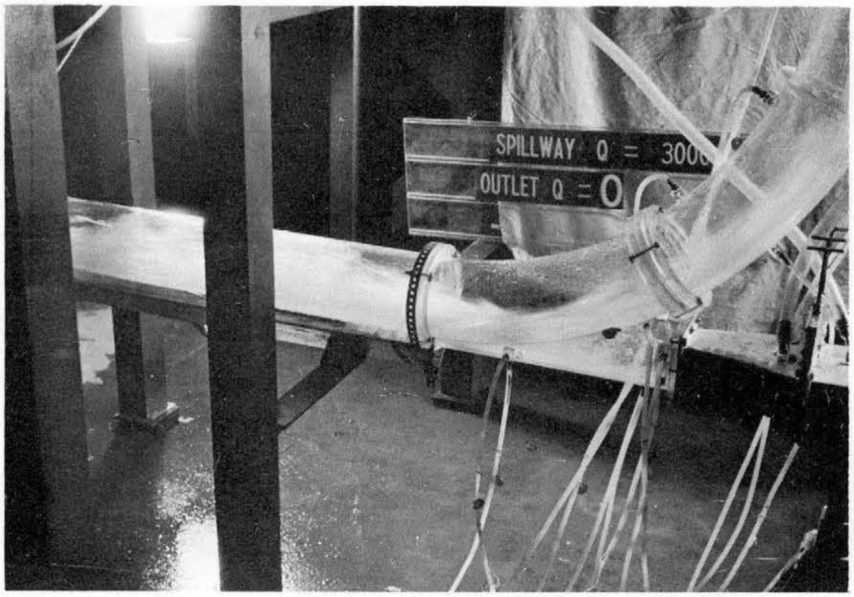


Fig. 92. Outlet Conduit 8 x 8 ft.
Spillway Q = 3000 c.f.s. Outlet Q = 0.

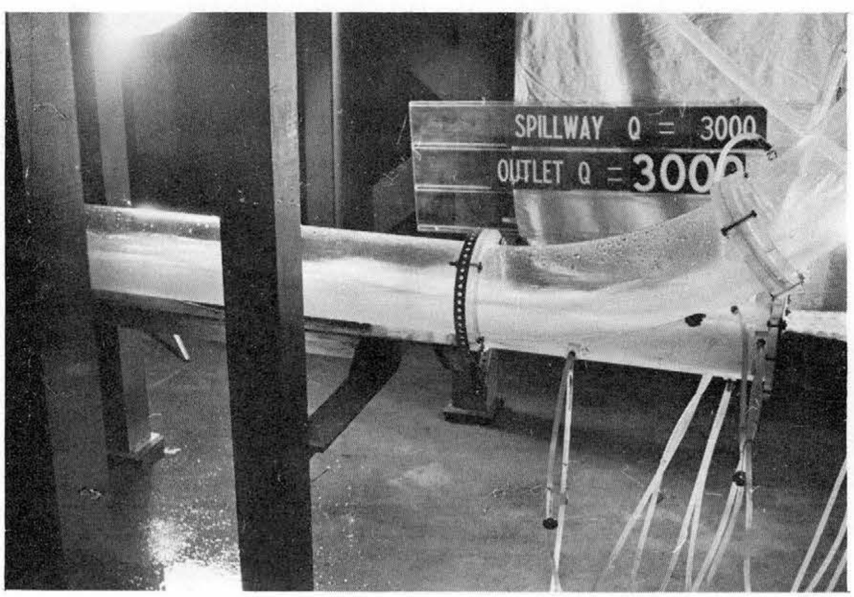


Fig. 93. Outlet Conduit 8 x 8 ft.
Spillway Q = 3000 c.f.s. Outlet Q = 3000 c.f.s.

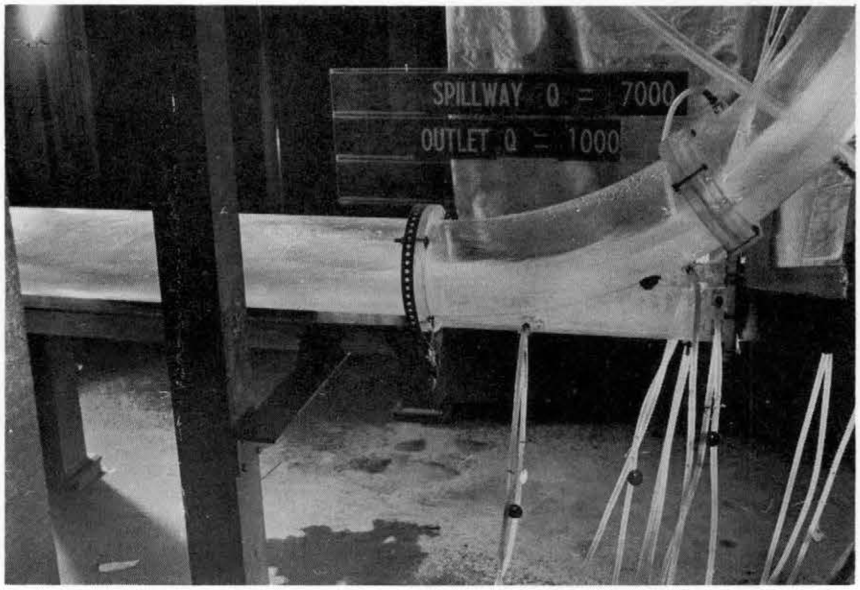


Fig. 94. Outlet Conduit 8 x 8 ft.
Spillway Q = 7000 c.f.s. Outlet Q = 1000 c.f.s.

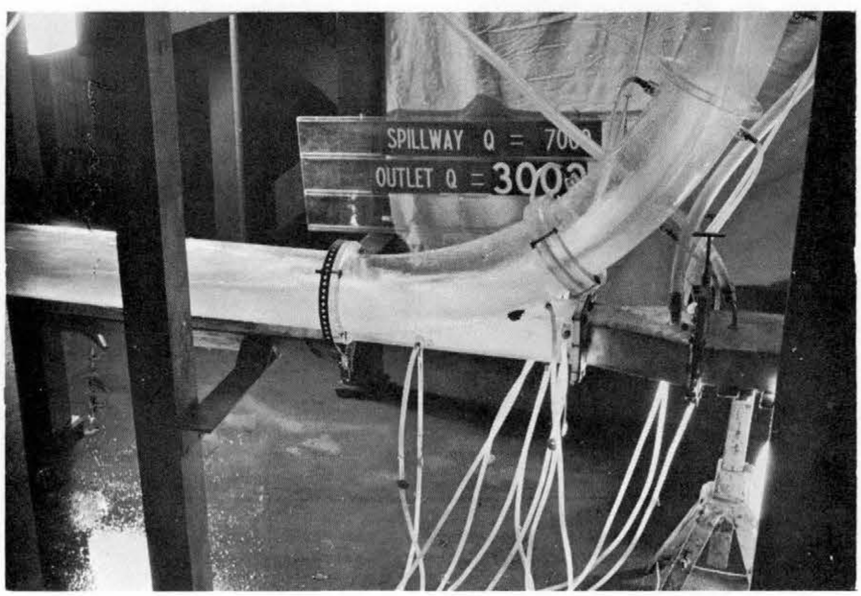


Fig. 95. Outlet Conduit 8 x 8 ft.
Spillway Q = 7000 c.f.s. Outlet Q = 3000 c.f.s.

Fig. 96. Outlet Conduit 8 x 8 ft.
Spillway Q = 11,500 c.f.s. Outlet Q = 1000 c.f.s.

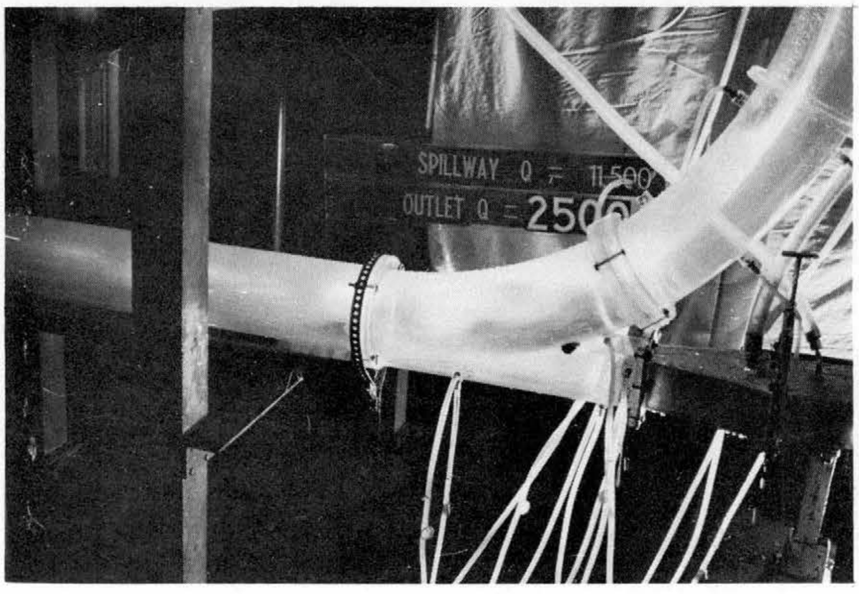
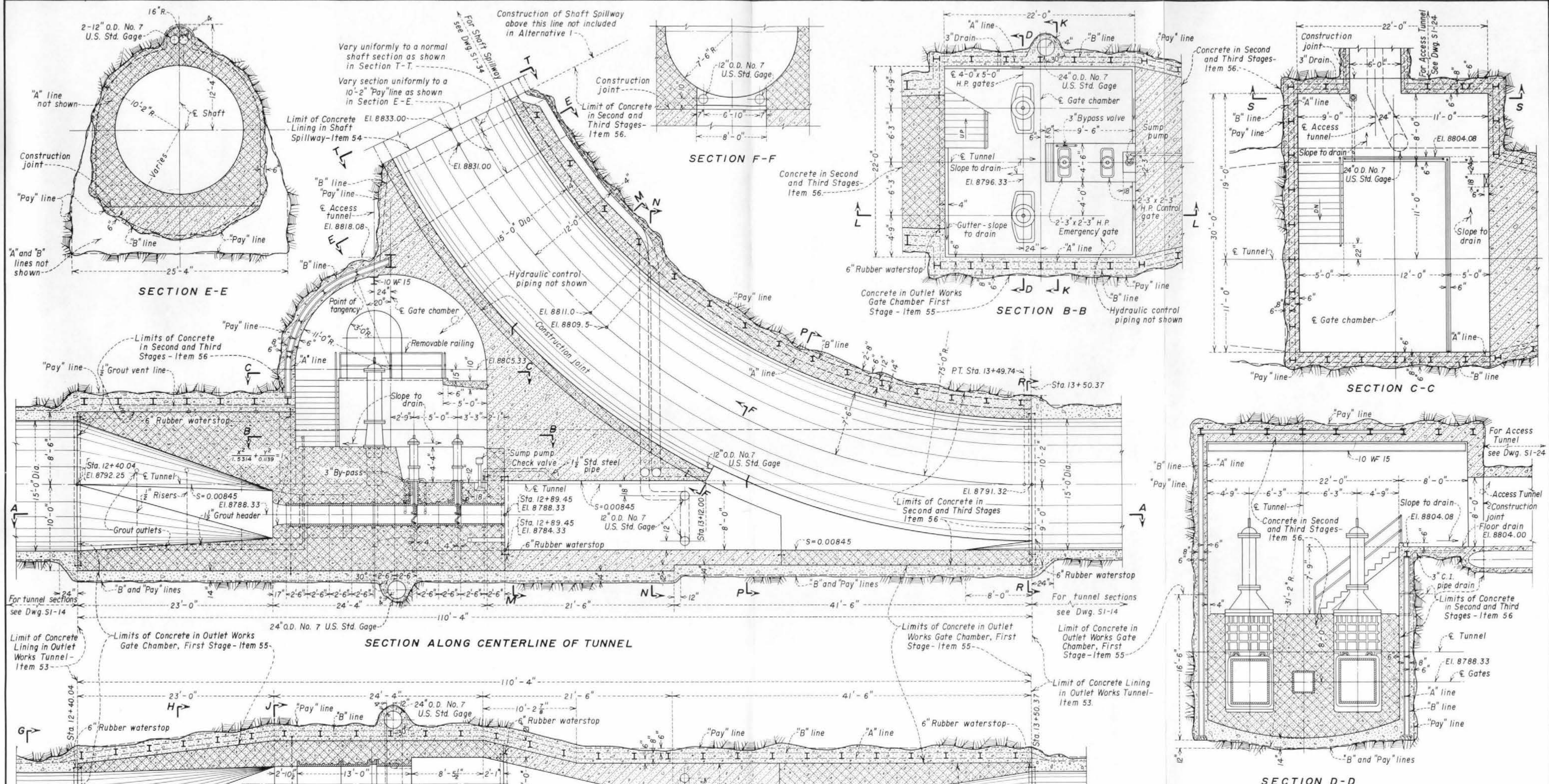
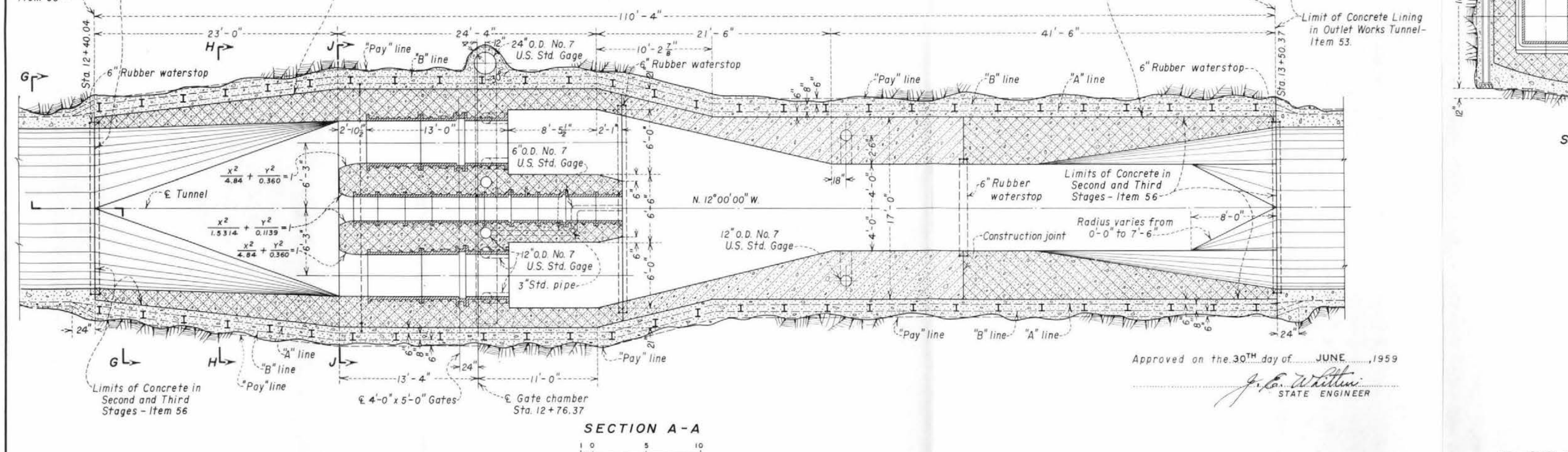


Fig. 97. Outlet Conduit 8 x 8 ft.
Spillway Q = 11,500 c.f.s. Outlet Q =2500 c.f.s.





Approved on the 30TH day of JUNE 1959
J. E. Whitten
STATE ENGINEER

NOTES

- Denotes first stage concrete
- Denotes second stage concrete
- Denotes third stage concrete

For General Structural Notes, see Drawing SI-3
For Gate Chamber Reinforcement, see Drawings SI-22 and SI-23.
For Outlet Works-Access Shaft, Tunnel and Gate Chamber Electrical Installation, see Drawing SI-29
For Outlet Works-Mechanical Installation, see Drawing SI-32
For Outlet Works-Miscellaneous Metalwork, see Drawing SI-37
Lagging behind steel supports not shown.

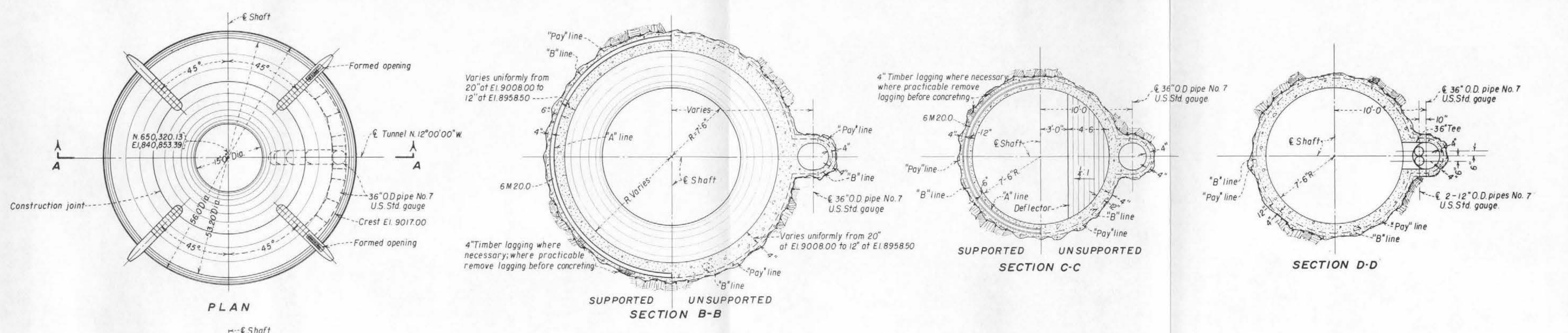
BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC., ENGINEERS
DILLON DAM
ALTERNATIVES 1, 2 AND 3
OUTLET WORKS GATE CHAMBER
PLAN AND SECTIONS

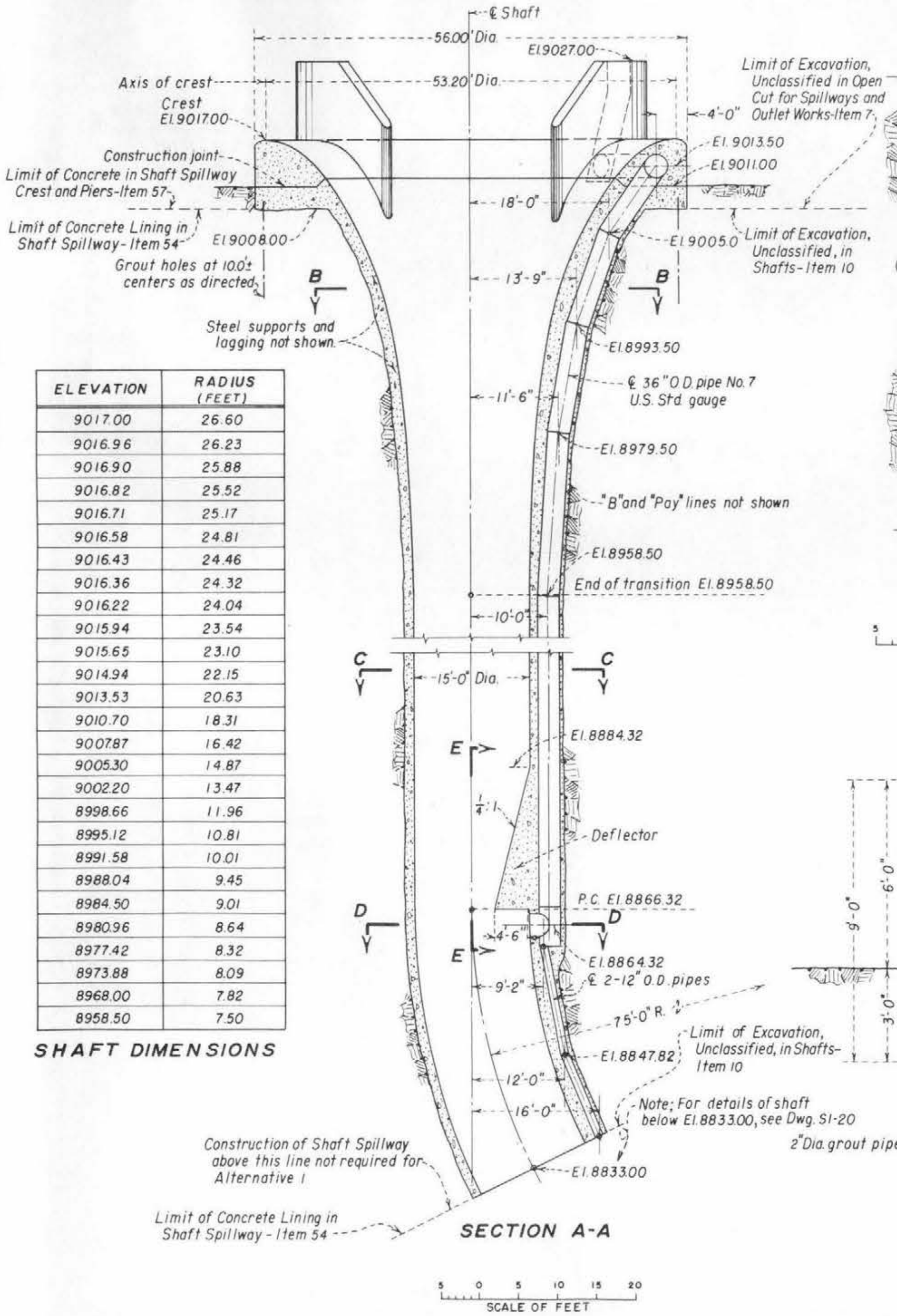
DRAWN J.C.E.		APPROVED	TIPTON AND KALMBACH, INC.
TRACED R.V.S.		APPROVED	BOARD OF WATER COMMISSIONERS
CHECKED D.G.T.			

C-930

TK35	DENVER, COLORADO, JUNE, 1959	SI-20

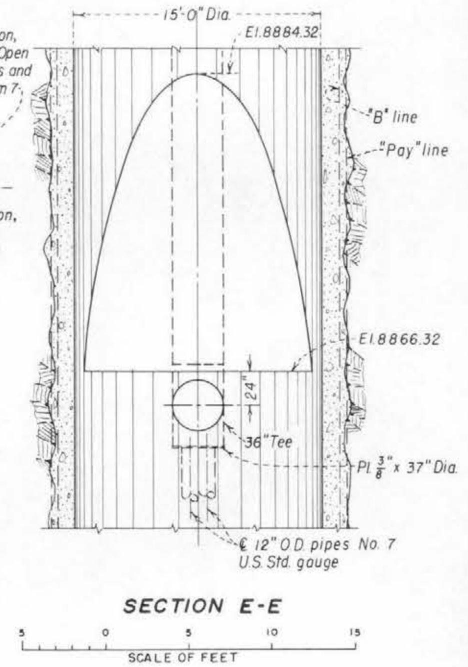
FIG.

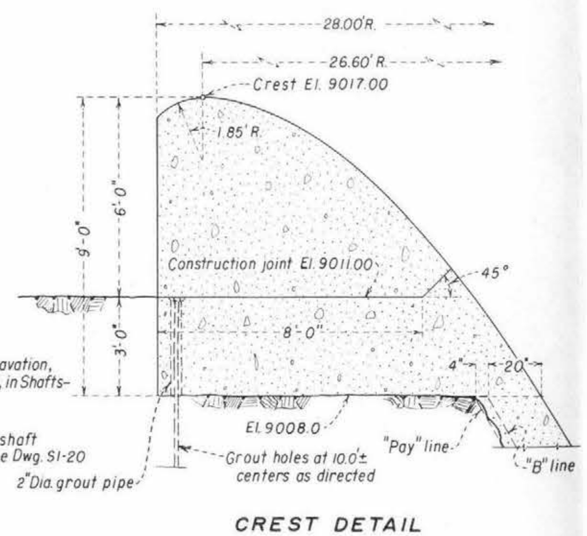


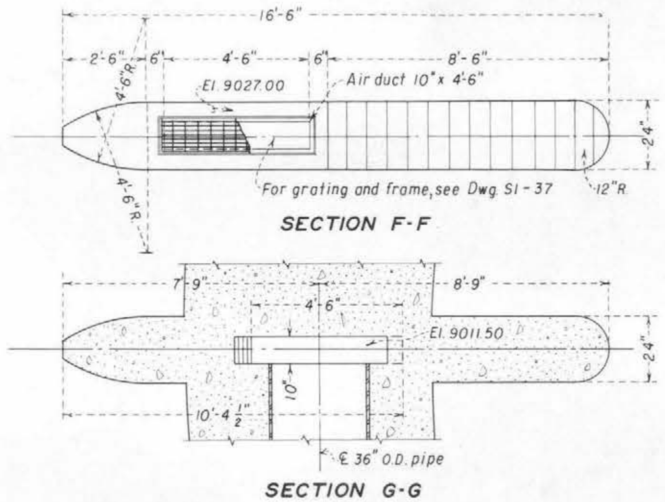


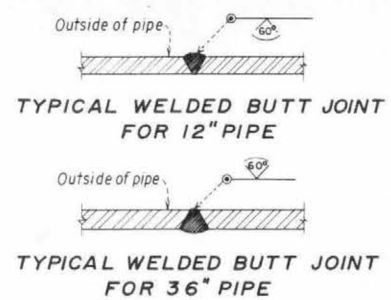
ELEVATION	RADIUS (FEET)
9017.00	26.60
9016.96	26.23
9016.90	25.88
9016.82	25.52
9016.71	25.17
9016.58	24.81
9016.43	24.46
9016.36	24.32
9016.22	24.04
9015.94	23.54
9015.65	23.10
9014.94	22.15
9013.53	20.63
9010.70	18.31
9007.87	16.42
9005.30	14.87
9002.20	13.47
8998.66	11.96
8995.12	10.81
8991.58	10.01
8988.04	9.45
8984.50	9.01
8980.96	8.64
8977.42	8.32
8973.88	8.09
8968.00	7.82
8958.50	7.50

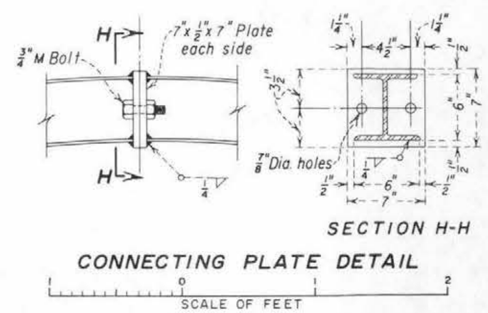
SHAFT DIMENSIONS

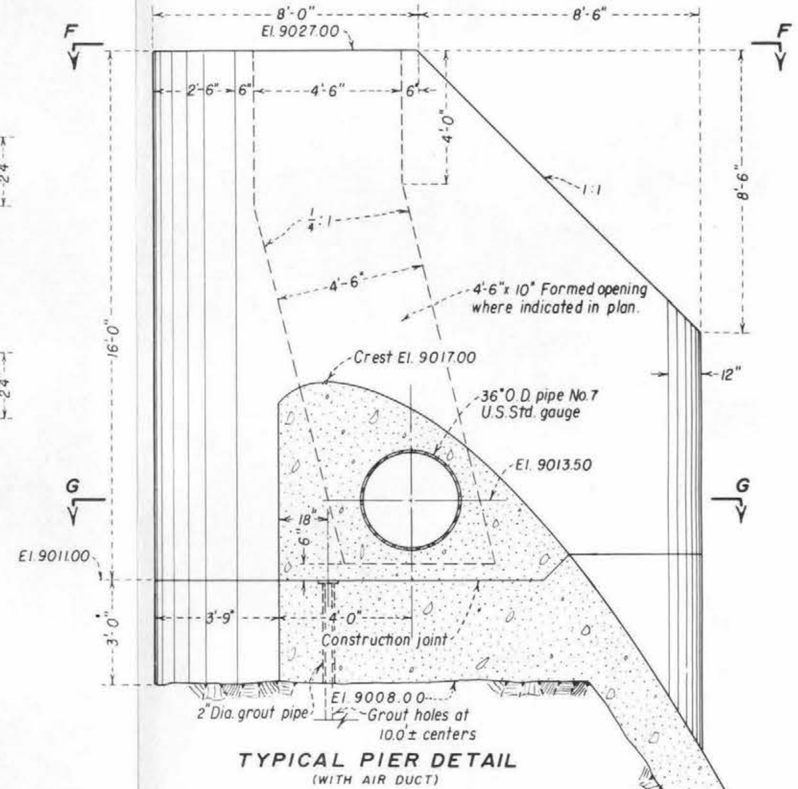












NOTES

For General Structural Notes, see Drawing SI-3.
For Gate Chamber Plan and Sections, see Drawing SI-20
For Plot Plan, West Abutment, see Drawing SI-13
For Shaft Spillway, Reinforcement, see Drawing SI-34
For Miscellaneous Metalwork, see Drawing SI-37

FIG. 99

BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC., ENGINEERS

DILLON DAM
ALTERNATIVES 2 AND 3
SHAFT SPILLWAY
PLAN AND SECTIONS

DRAWN L.I.V. — APPROVED TIPTON AND KALMBACH, INC.
TRACED G.Z.
CHECKED K.D.H. — APPROVED BOARD OF WATER COMMISSIONERS

Approved on the 30TH day of JUNE, 1959
J. E. Whitten
STATE ENGINEER

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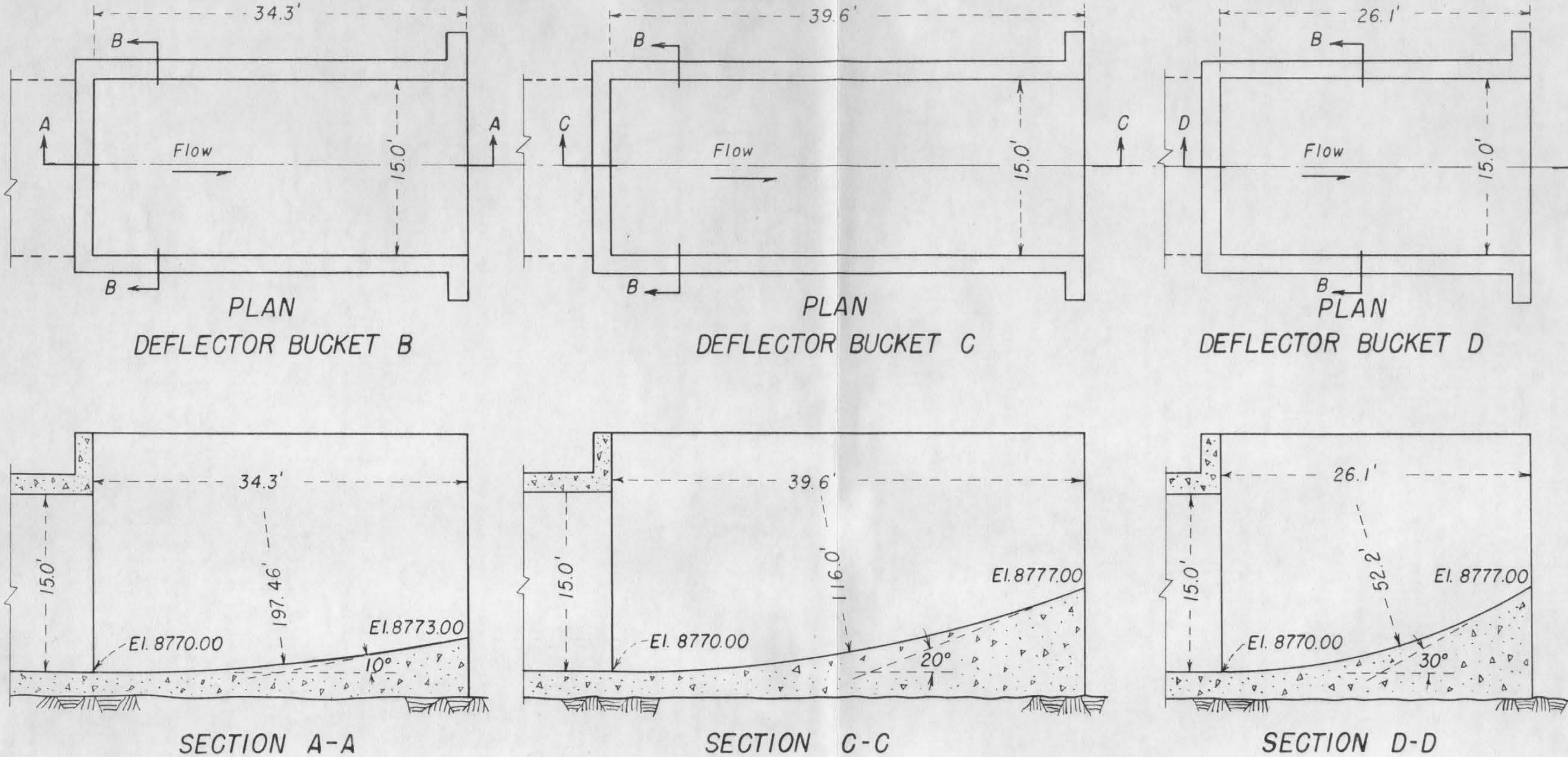
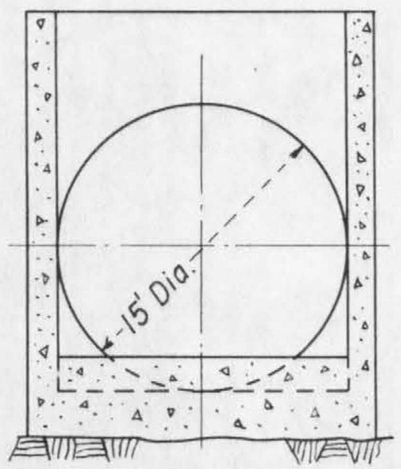




SECTION B-B

NOTES

Dimensions are shown in terms of the prototype.

Structural details are not intended.

10 0 20 ft.

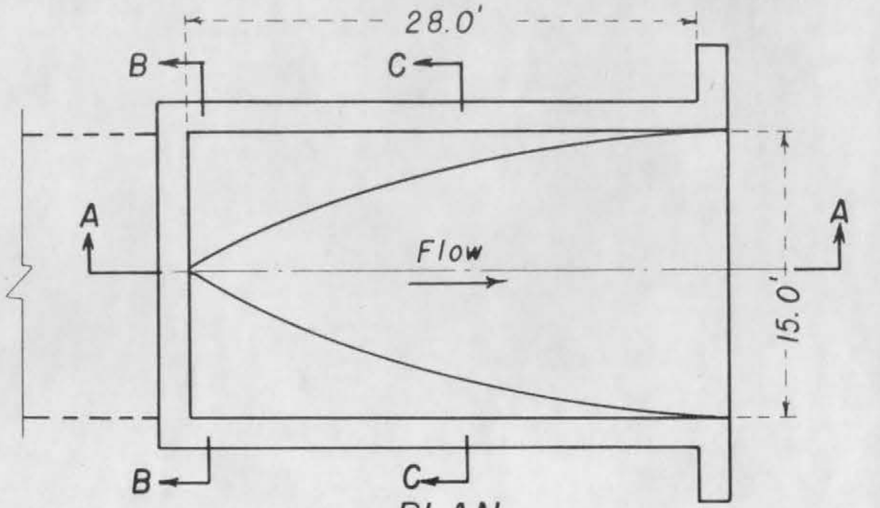
SCALE

COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION
MODEL STUDY
DILLON DAM SPILLWAY

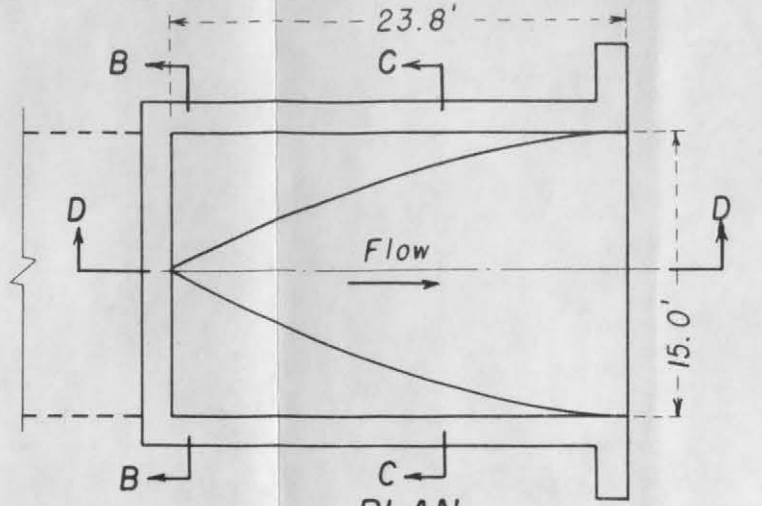
DEFLECTOR BUCKETS B, C & D PLANS AND SECTIONS

Drawn by B.H.B. Traced by H.A. M.C.C. Checked by S.K.	Approved S. Karaki

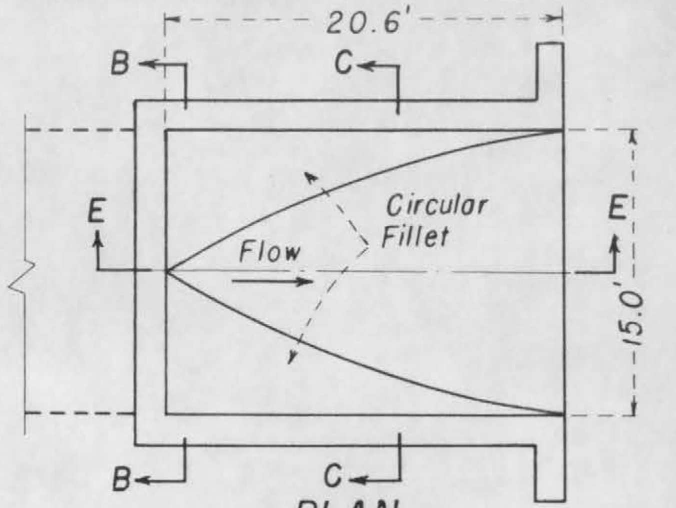
2002	JUNE 1959	FIG. 100



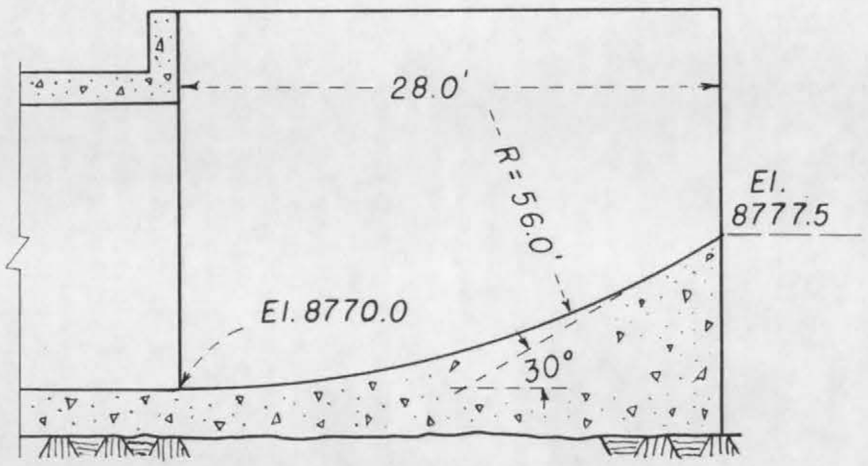
PLAN
DEFLECTOR BUCKET E



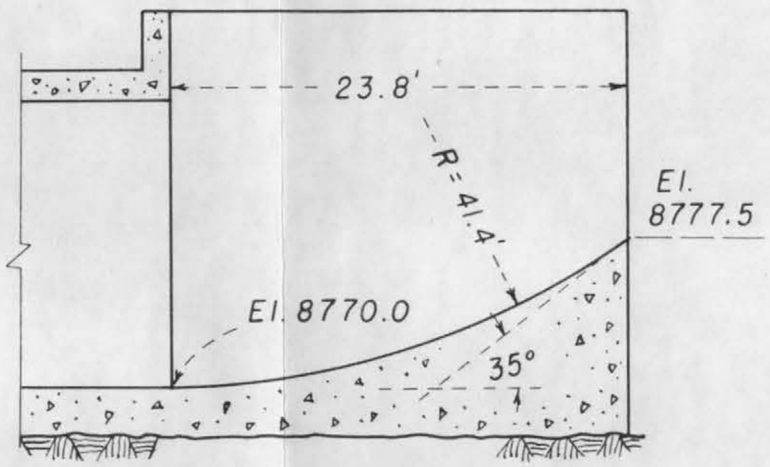
PLAN
DEFLECTOR BUCKET F



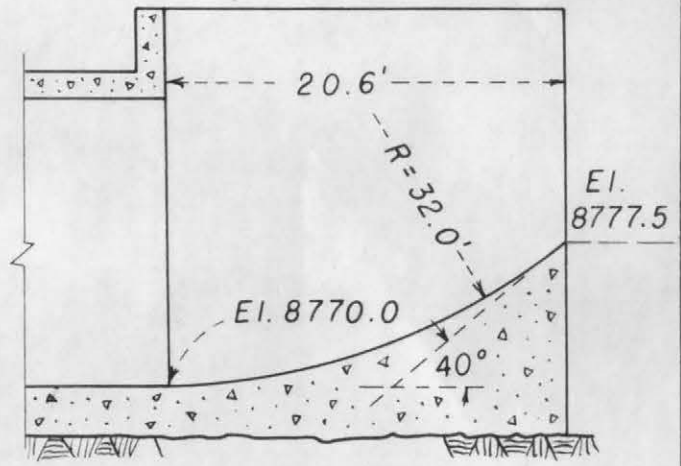
PLAN
DEFLECTOR BUCKET G



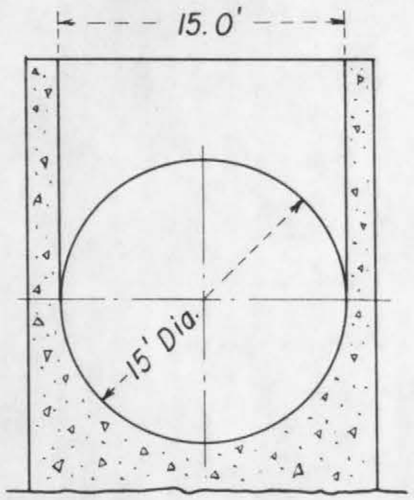
SECTION A-A



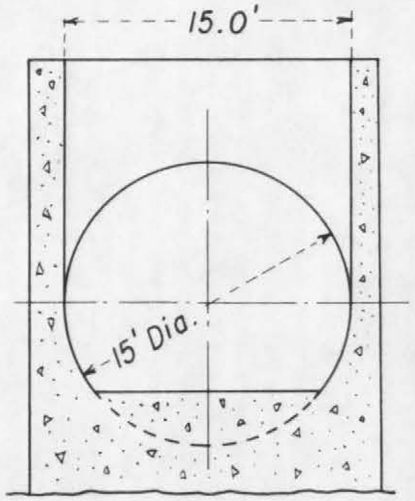
SECTION D-D



SECTION E-E



SECTION B-B



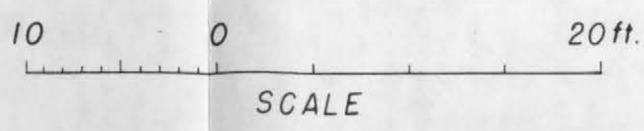
SECTION C-C

NOTES

Flip Buckets E, F, and G are flat bottomed with a constant radius fillet of 7.5 ft.

Dimensions are in terms of prototype.

Drawings are made to indicate dimensions only. Structural details are not intended.



COLORADO STATE UNIVERSITY
RESEARCH FOUNDATION

MODEL STUDY
DILLON DAM SPILLWAY

DEFLECTOR BUCKETS E, F, & G PLAN AND SECTIONS

Drawn by ___ B.H.B. Traced by ___ H.A. Checked by __ S.K.	Approved S. Karaki

2002	JUNE 1959	FIG. 101

Fig. 102. Q = 1000 c.f.s.
Flow over the end sill of Deflector Bucket F.
Height of sill = 7.5 ft.

Fig. 103 Jet Trajectory of Deflector Bucket F.
Discharge = 5000 c.f.s. through
Spillway. Numbers on Channel wall
are distances from the tunnel portal.

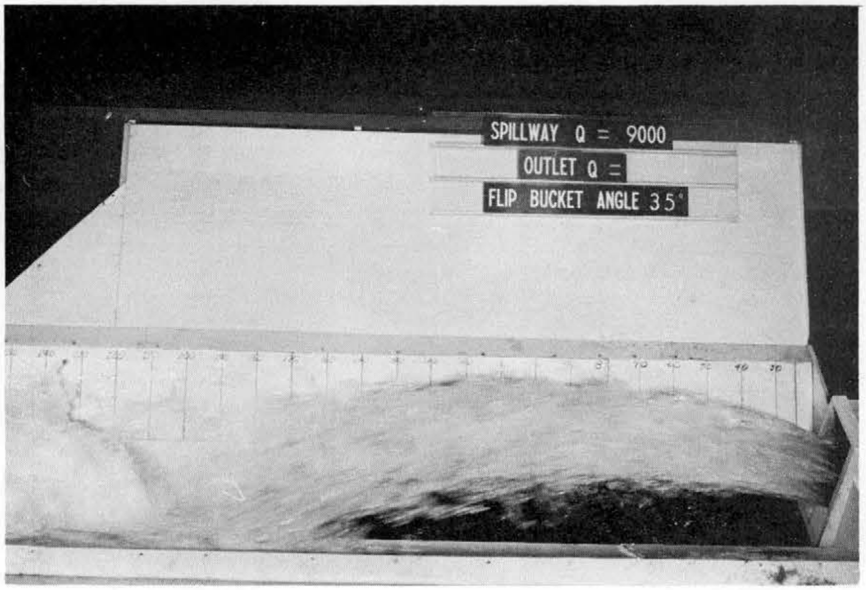


Fig. 104. Jet Trajectory of Deflector Bucket F.
Discharge = 9000 c.f.s. through spillway.

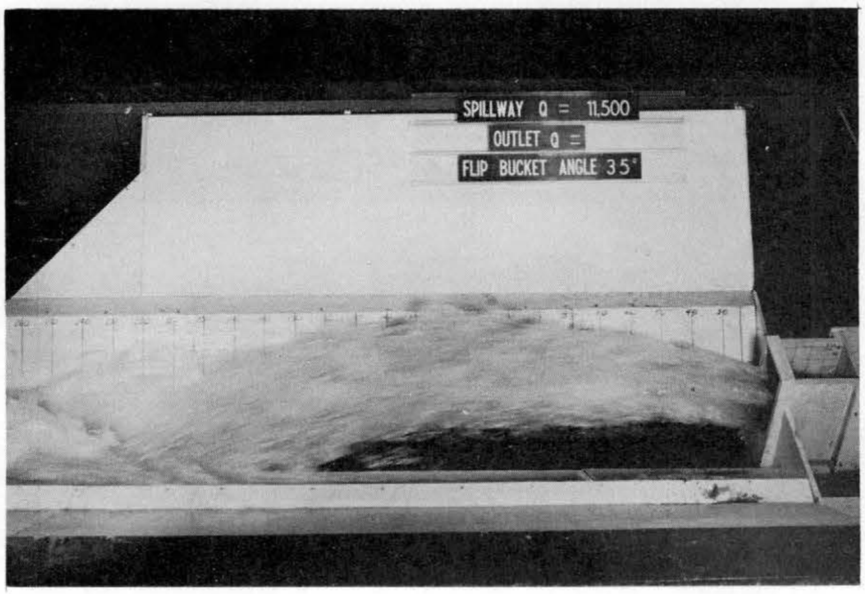
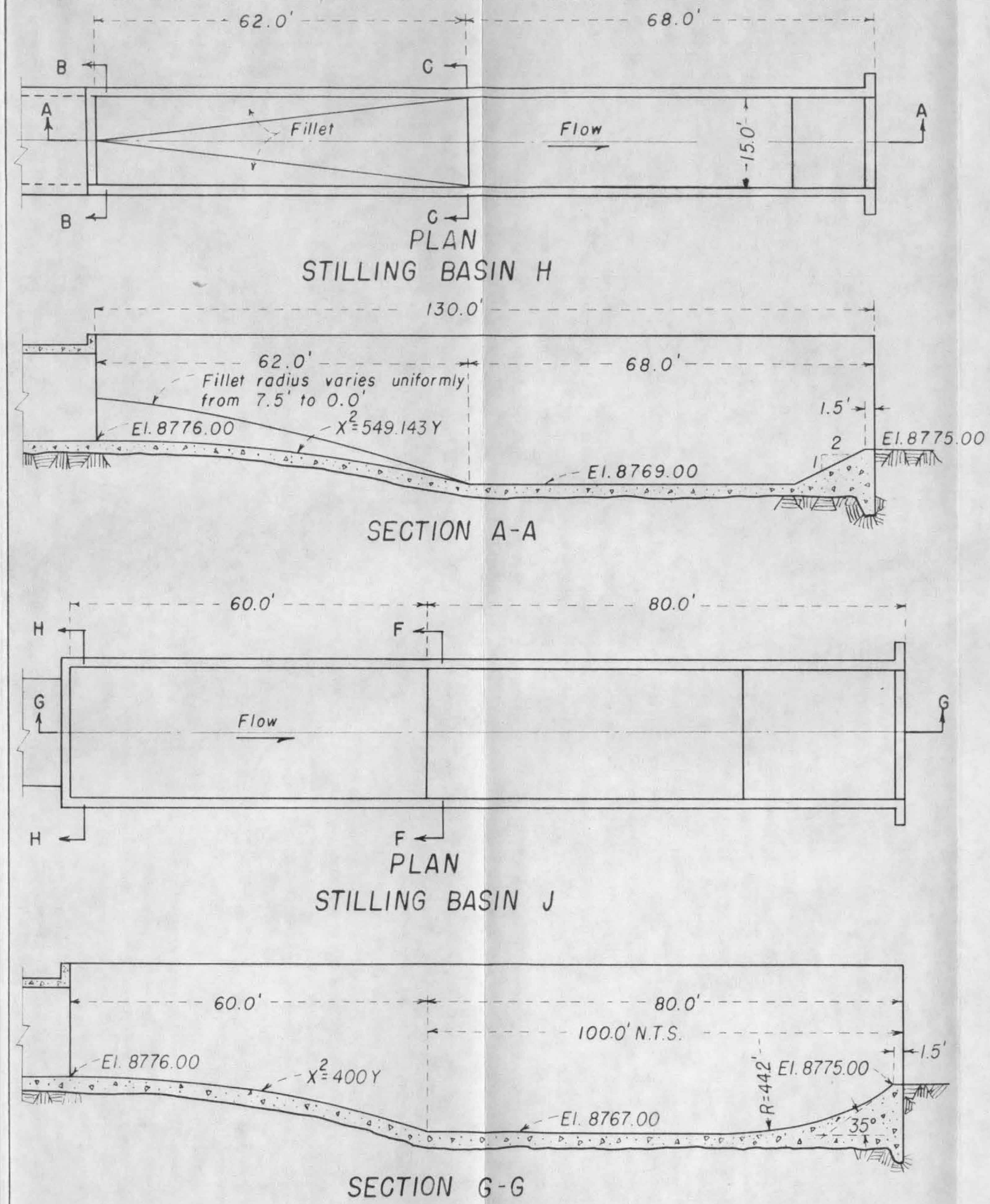
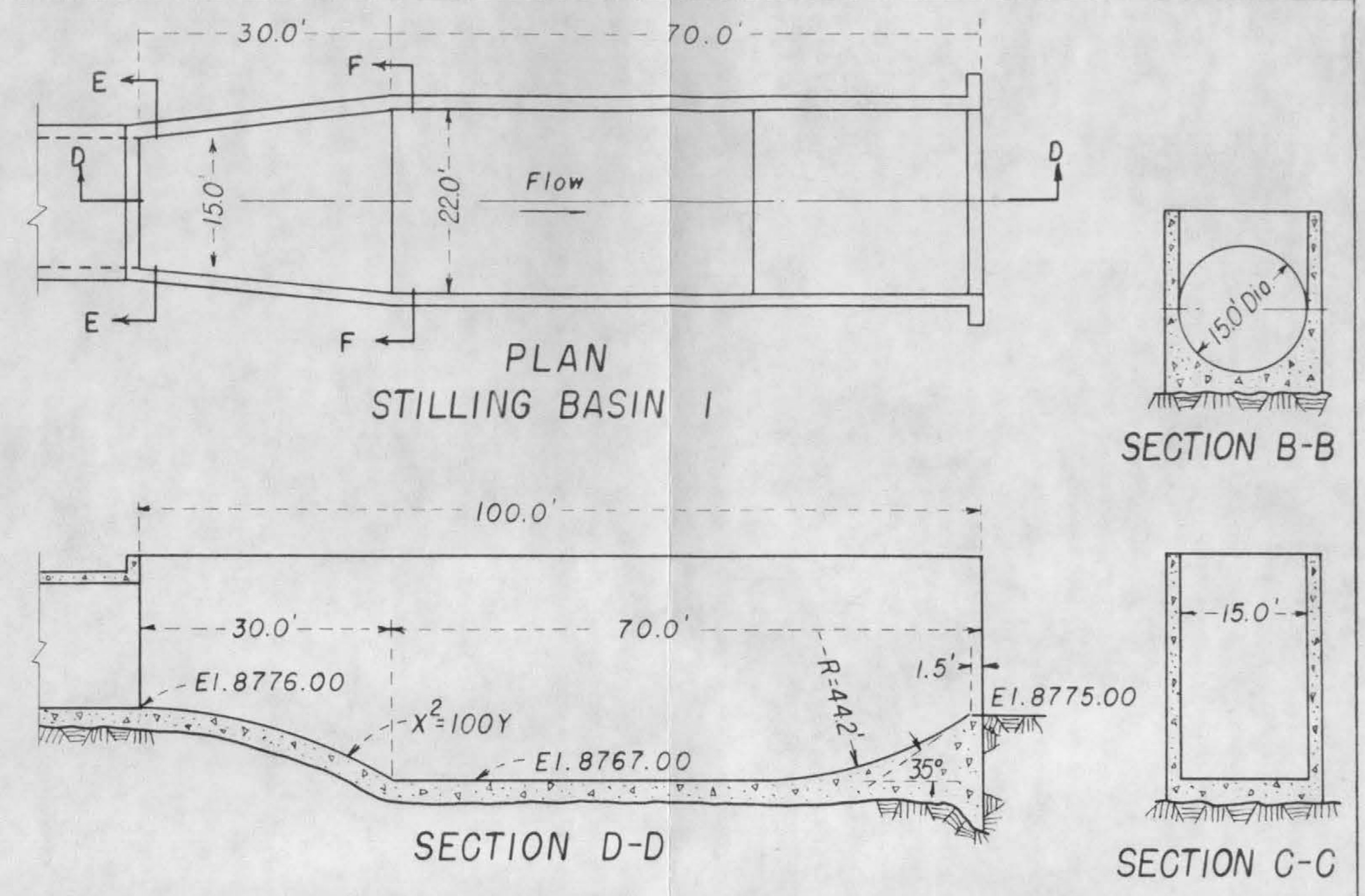
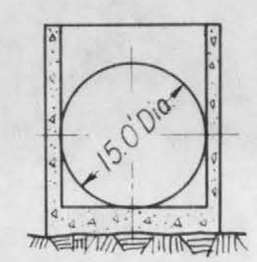
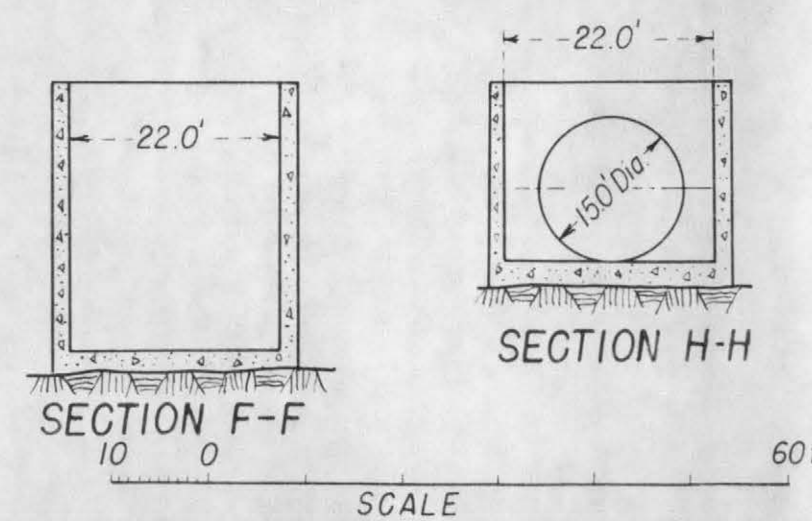


Fig. 105. Jet Trajectory of Deflector Bucket F.
Discharge = 11,500 c.f.s. through spillway.









NOTES

Stilling Basin H shows a variable radius fillet on the trajectory. This fillet was removed in later testing to improve the shape of the water jet formed at the end sill. A sweeping curve was also used in place of the 2:1 sloped end sill.

Stilling Basin J shows a basin length of 80 ft. A basin length of 100 ft. was also tested.

Dimensions are in terms of prototype.

COLORADO STATE UNIVERSITY
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MODEL STUDY
DILLON DAM SPILLWAY

STILLING BASINS H, I & J
PLAN AND SECTIONS

Drawn by B.H.B. Traced by H.A. M.C.C. Checked by S. K.	Approved S. Karaki
2002	JUNE 1959
	FIG. 106

Fig. 107. Q = 5000 c.f.s. Stilling Basin H.
Note irregular jet formation and high central fin.

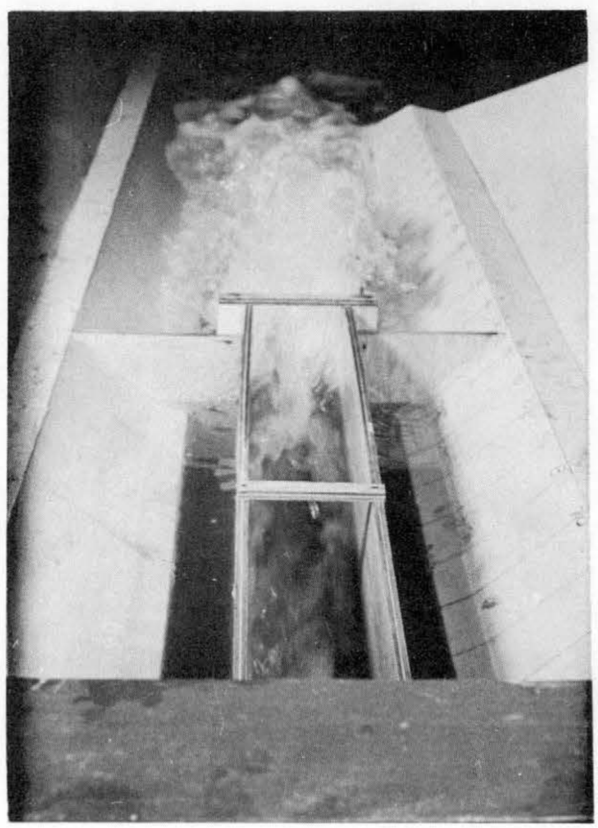


Fig. 108. Q = 5000 c.f.s. Downstream View.
Note standing waves meet at the end sill.
End sill slopes 2:1.

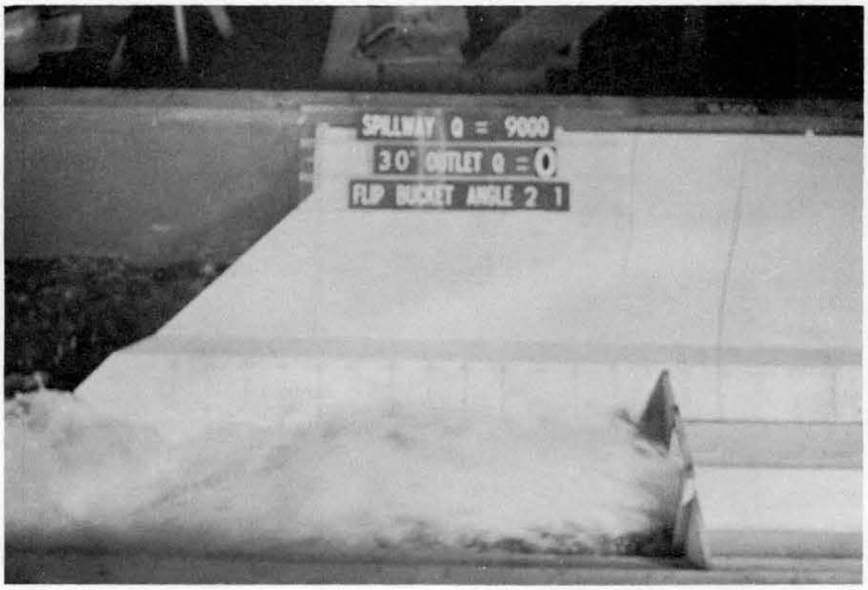


Fig. 109. Q = 9000 c.f.s. Stilling Basin H.

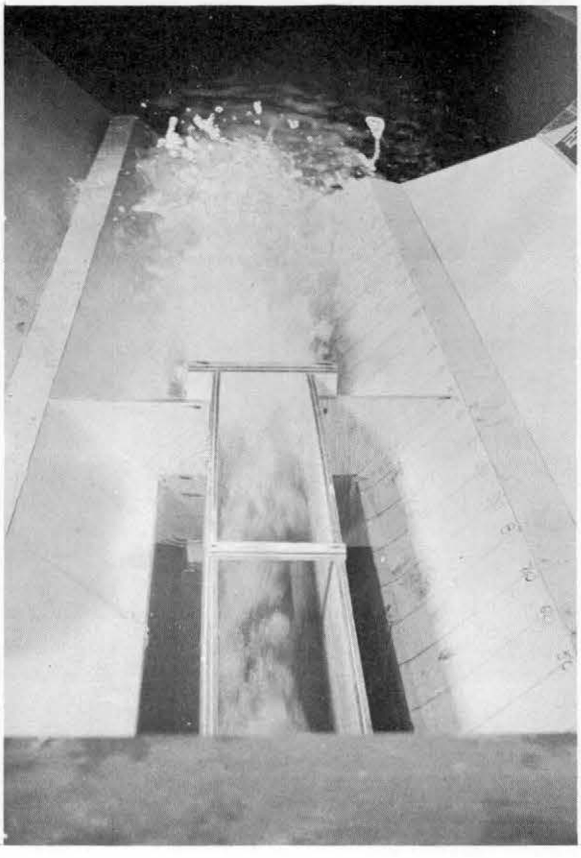
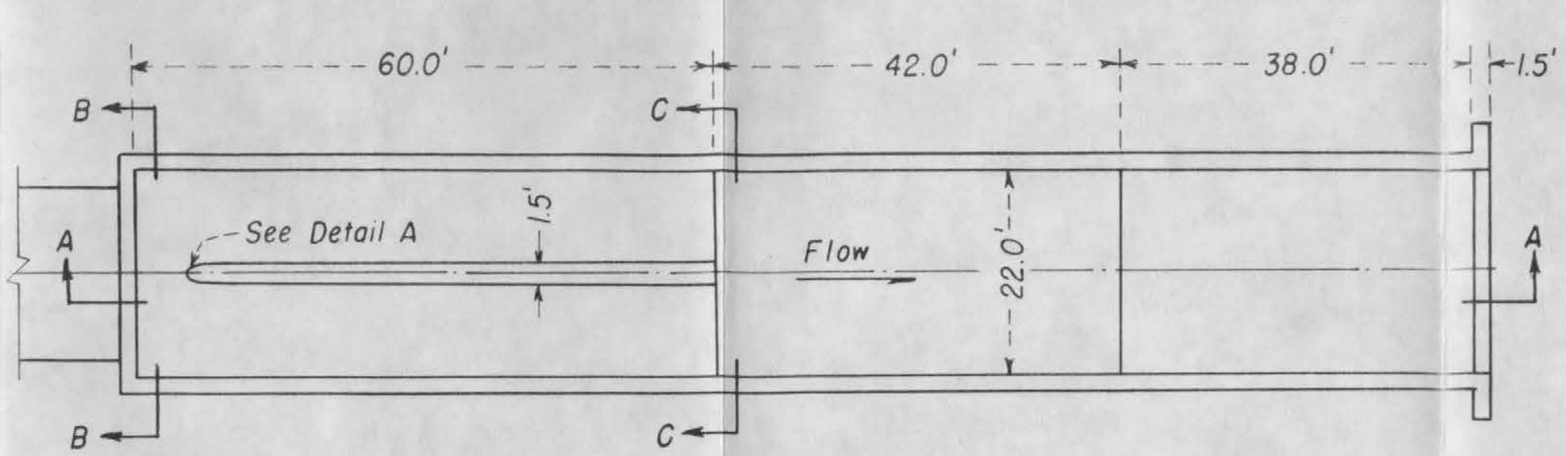
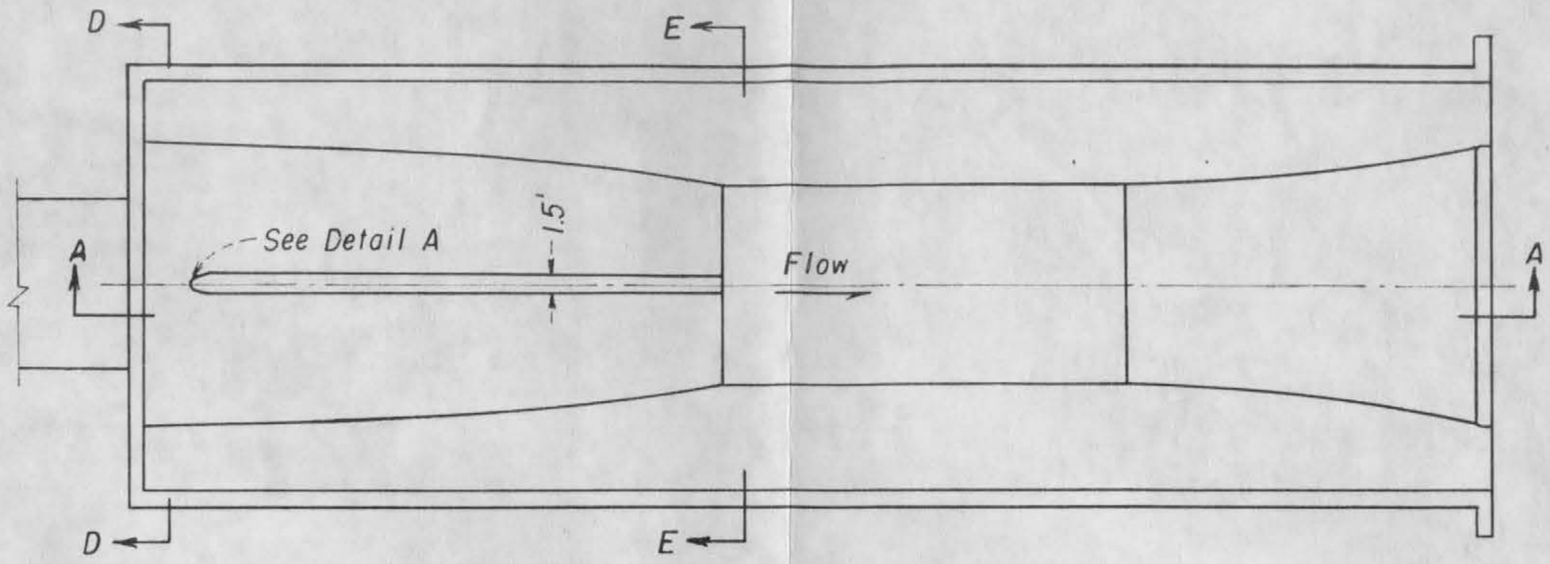


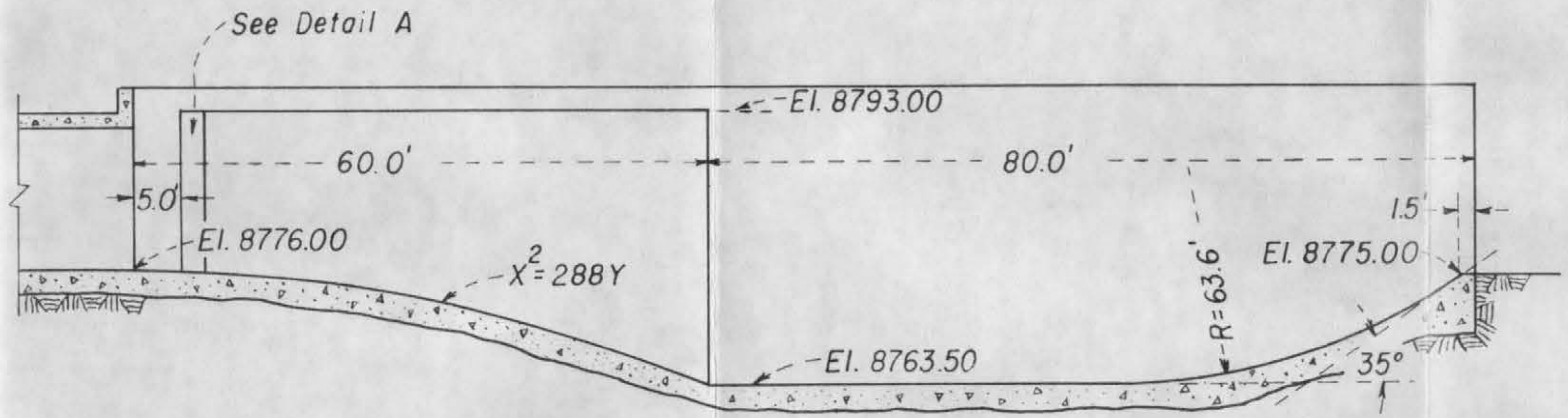
Fig. 110. Q = 9000 c.f.s. Downstream View.



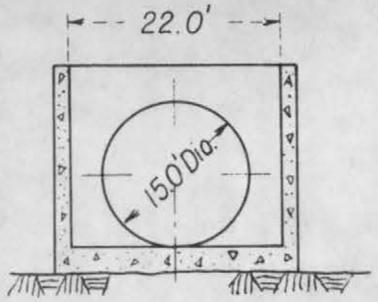
PLAN
STILLING BASIN K



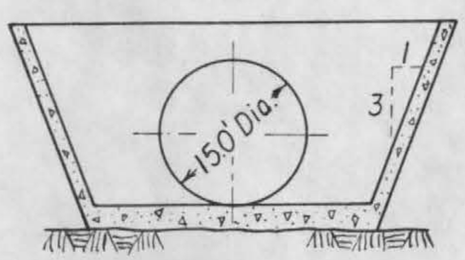
PLAN
STILLING BASIN L



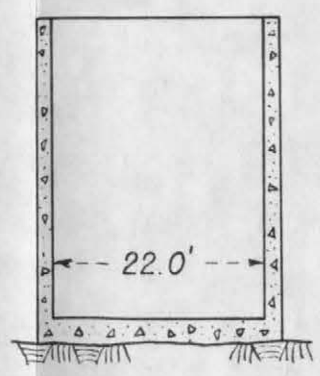
SECTION A-A



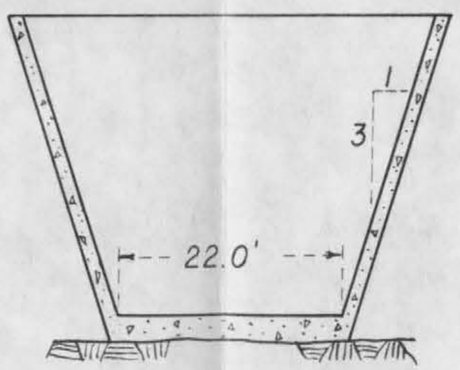
SECTION B-B



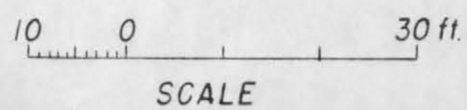
SECTION D-D

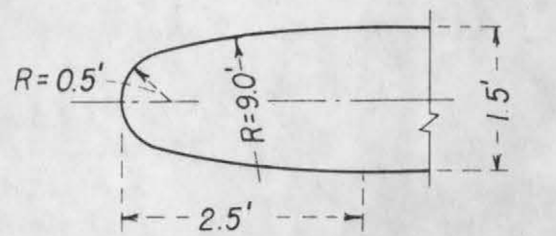


SECTION C-C

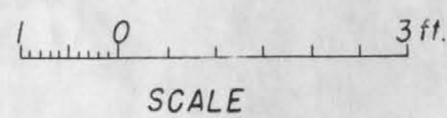


SECTION E-E





DETAIL A



NOTES

Stilling basin K was the recommend stilling basin. For the prototype, however, a cut and cover conduit section was used for the initial 60' ft. of length.

Stilling basin L was an attempt for economy of the gravity walls.

All dimensions shown are in terms of prototype. Construction details are not intended.

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MODEL STUDY
DILLON DAM SPILLWAY

STILLING BASINS K & L PLAN AND SECTIONS

Drawn by B.H.B. Traced by H.A. M.C.C. Checked by S. K.	Approved S. Karaki

2002	JUNE 1959	FIG. 111

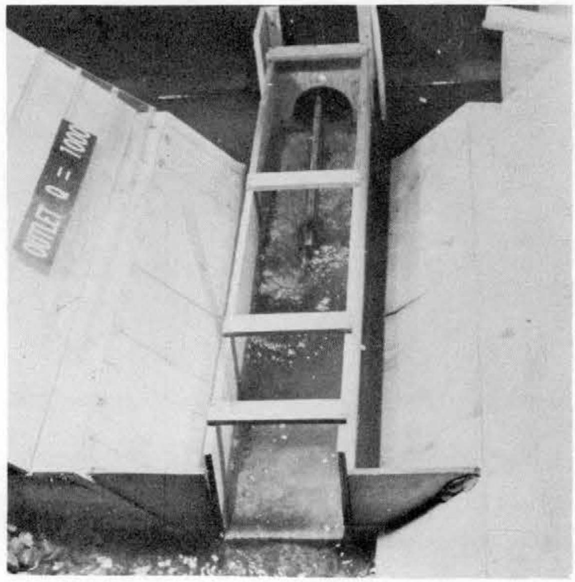


Fig. 112. Flow in Recommended Stilling Basin.
Q = 1000 c.f.s.

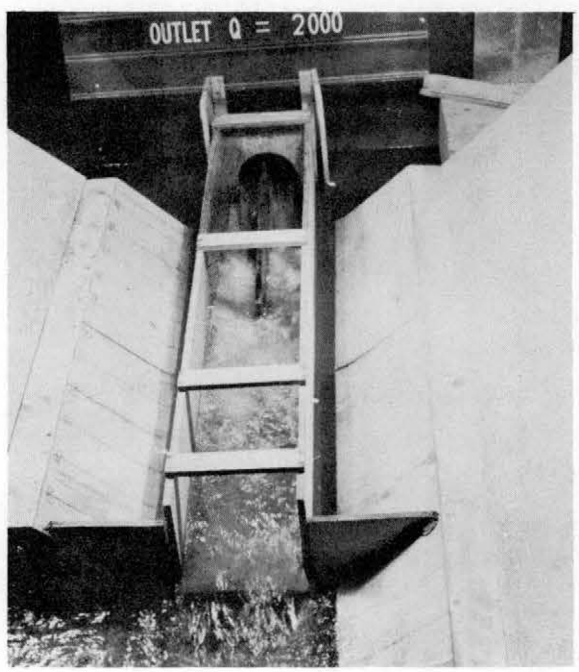


Fig. 113. Flow in Recommended Stilling Basin.
Q = 2000 c. f. s.

Fig. 114. Flow in Recommended Stilling Basin. Q = 3000 c.f.s.

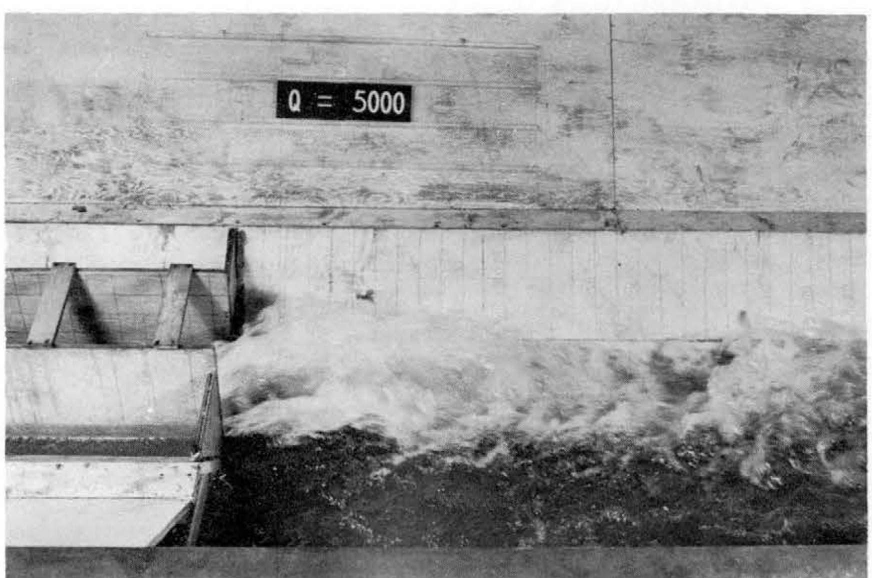


Fig. 115. Flow in Recommended Basin. Q = 5000 c.f.s.

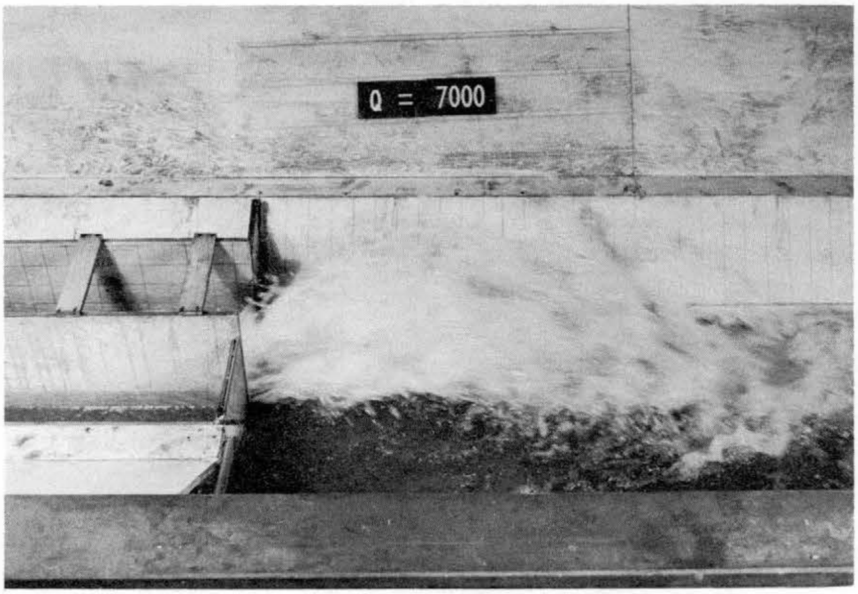


Fig. 116. Flow in Recommended Basin.
Q = 7000 c.f.s.

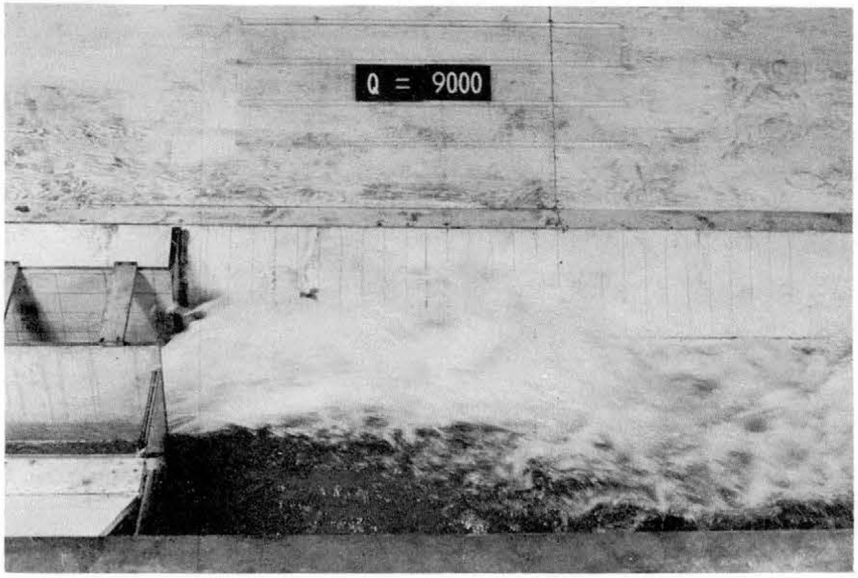


Fig. 117. Flow in Recommended Basin.
Q = 9000 c.f.s.

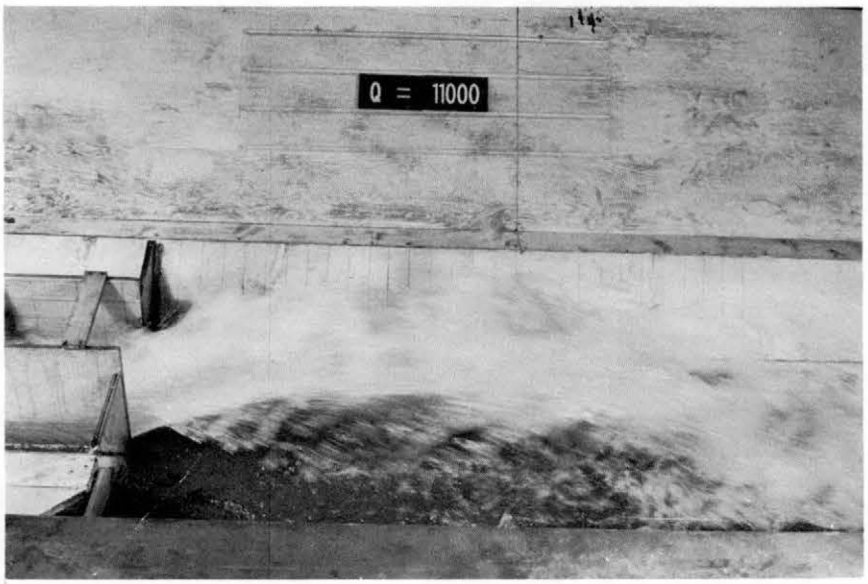


Fig. 118. Flow in Recommended Basin.
Q = 11,000 c.f.s.

Fig. 120. Recommended Stilling Basin.
5 Minute Scour Test.
Q = 7000 c.f.s.

Fig. 121. Recommended Stilling Basin.
5 Minute Scour Test.
Q = 9000 c.f.s.

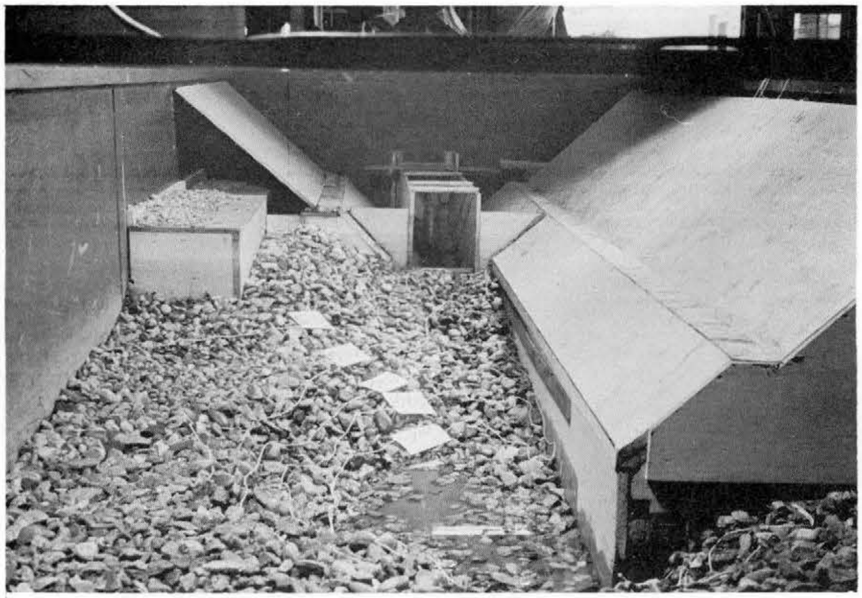


Fig. 122. Recommended Stilling Basin.
5 Minute Scour Test.
Q = 11,000 c.f.s.

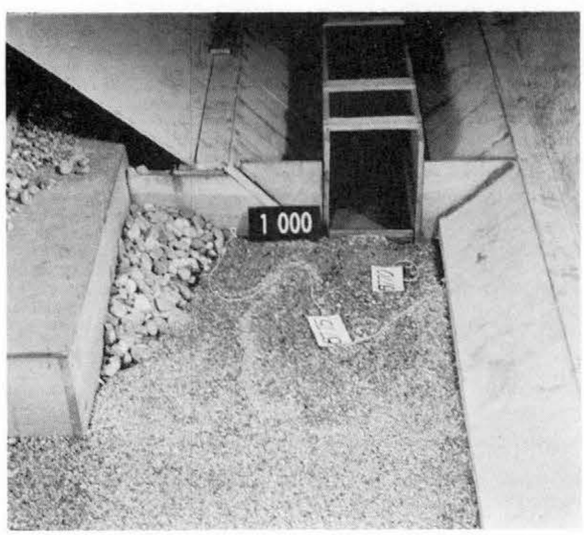


Fig. 123. Recommended Stilling Basin.
30 Minute Scour Test.
Q = 1000 c.f.s.

Fig. 124. Recommended Stilling Basin.
30 Minute Scour Test, Q = 2000 c.f.s.

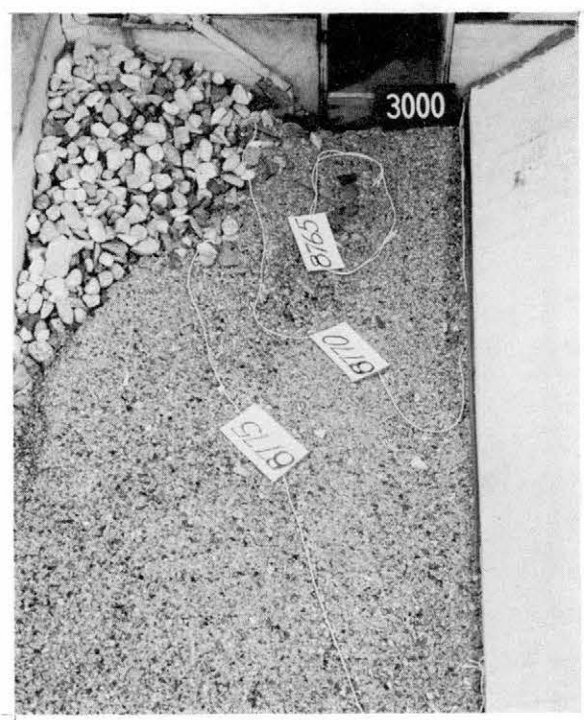
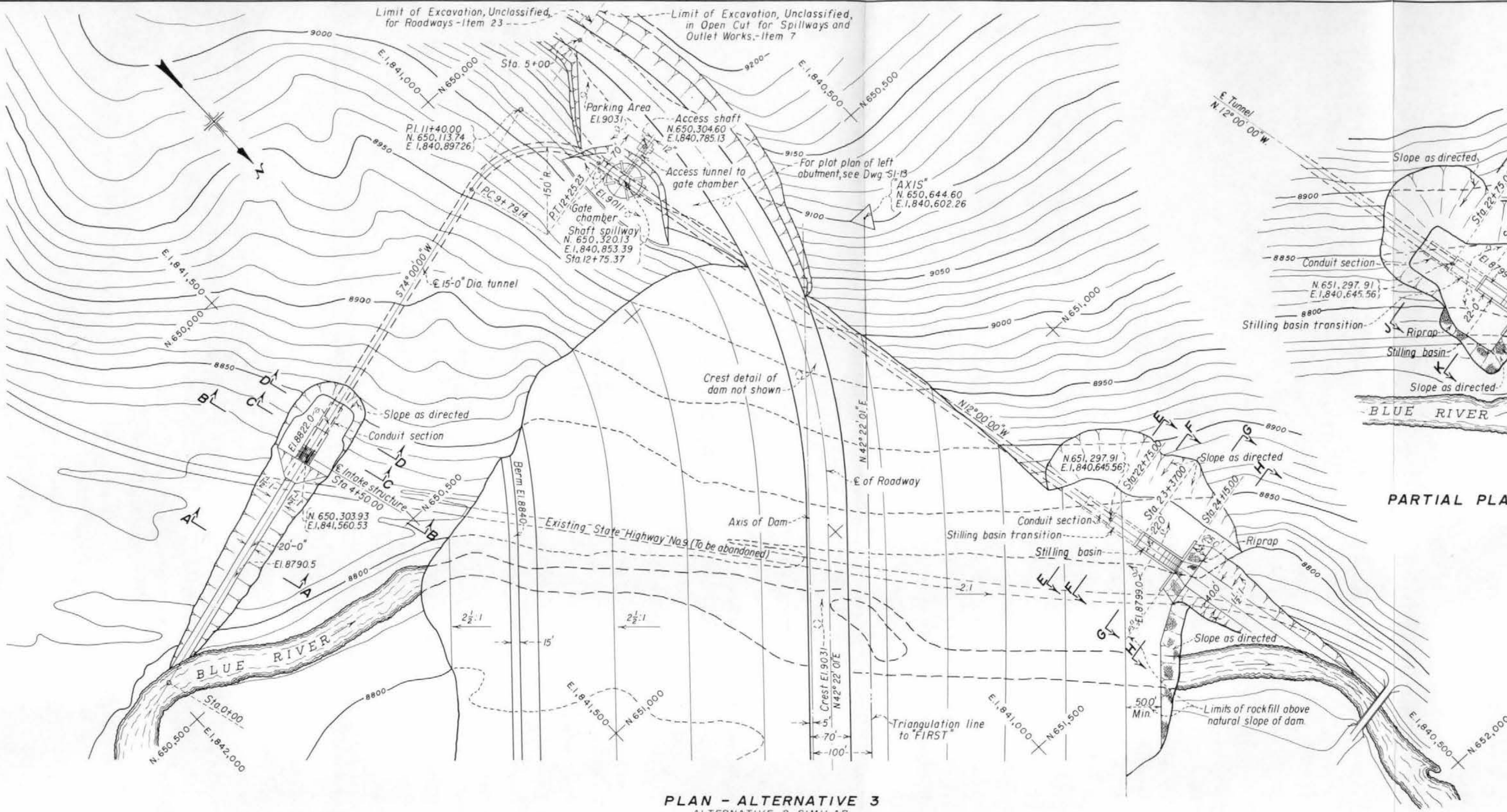


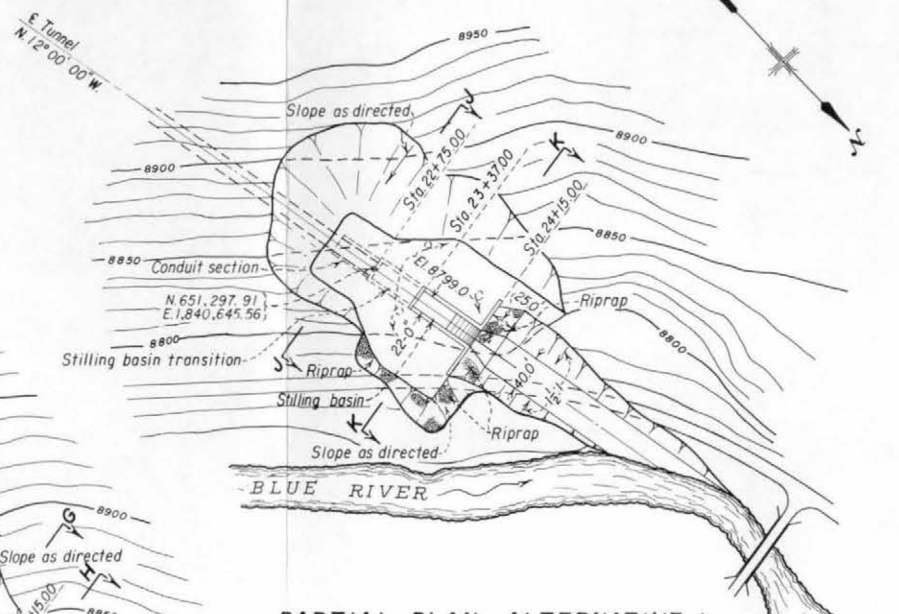
Fig. 125. Recommended Stilling Basin.
30 Minute Scour Test. Q = 3000 c.f.s.



PLAN - ALTERNATIVE 3

ALTERNATIVE 2 SIMILAR

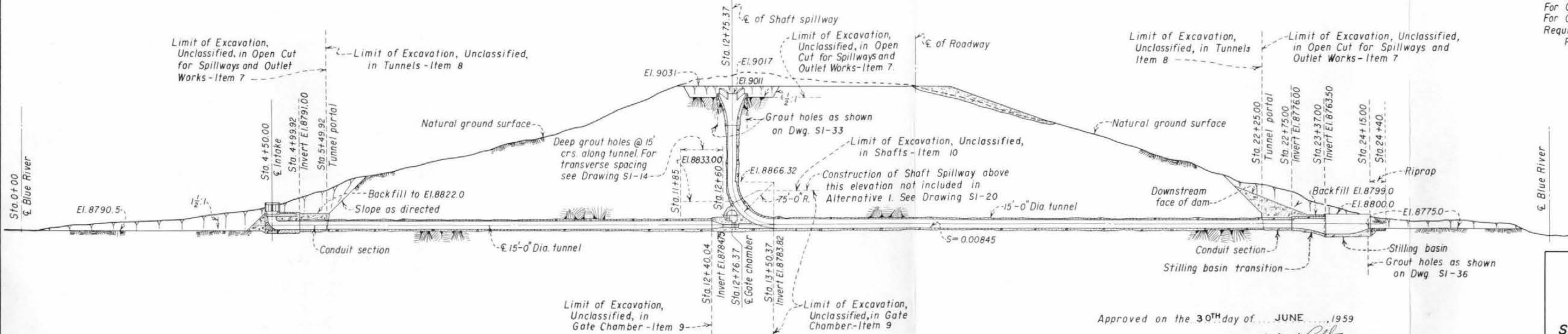
SCALE OF FEET



PARTIAL PLAN - ALTERNATIVE I

NOTES

For General Plans and Profiles, see Dwgs. SI-3, SI-5, and SI-6.
For Shaft Spillway and Outlet Works Sections, see Dwg. SI-12.
For Typical Tunnel Sections, see Dwg. SI-14.
For Outlet Works Intake Plan and Sections, see Dwg. SI-15.
For Shaft Spillway Plan and Sections, see Dwg. SI-33.
For Tunnel Portals and Conduits, see Dwg. SI-19.
For Outlet Works Control House, see Dwg. SI-25.
For Outlet Works Gate Chamber, see Dwg. SI-20.
For Outlet Works Stilling Basin Transition, see Dwg. SI-35
For Outlet Works Stilling Basin, see Dwg. SI-36.
Required Stations of Portals may vary from assumed Portal Stations shown.



PROFILE ALONG ℄ OF OUTLET WORKS

Approved on the 30TH day of JUNE 1959

J. C. Whitten
STATE ENGINEER

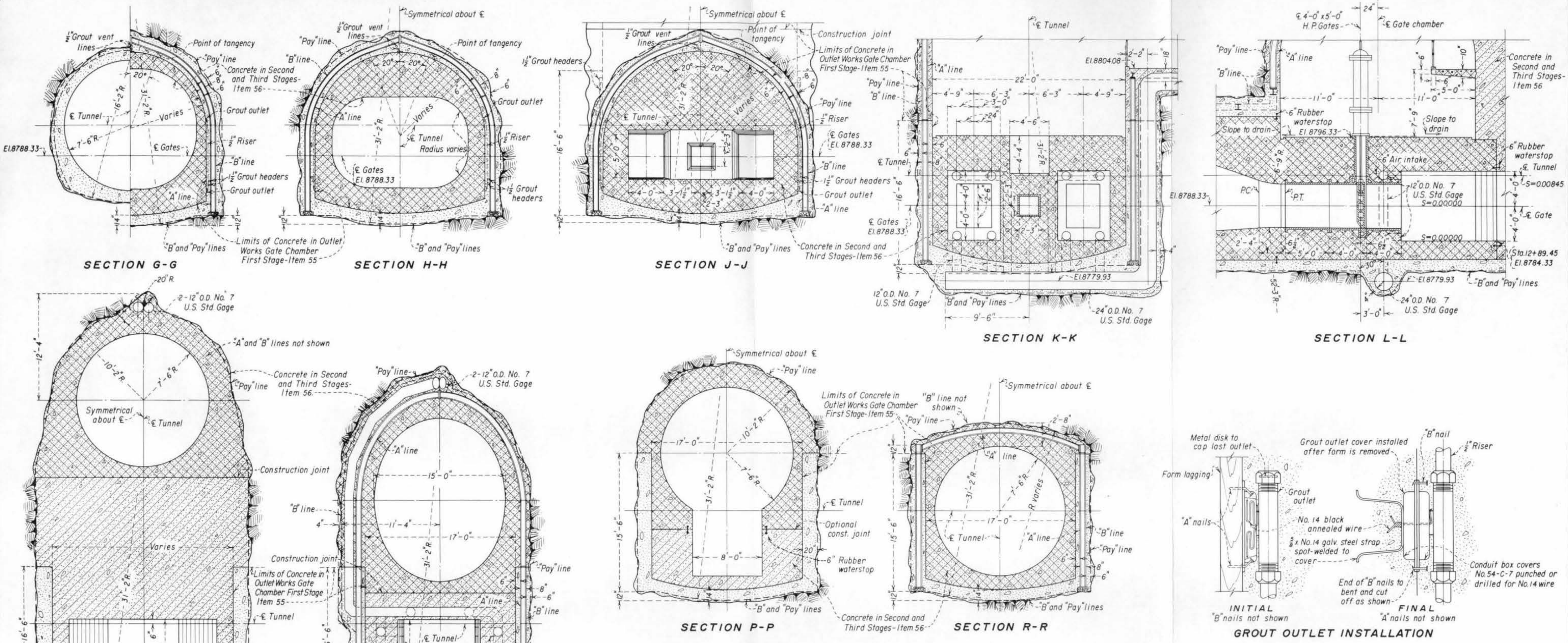
BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC., ENGINEERS

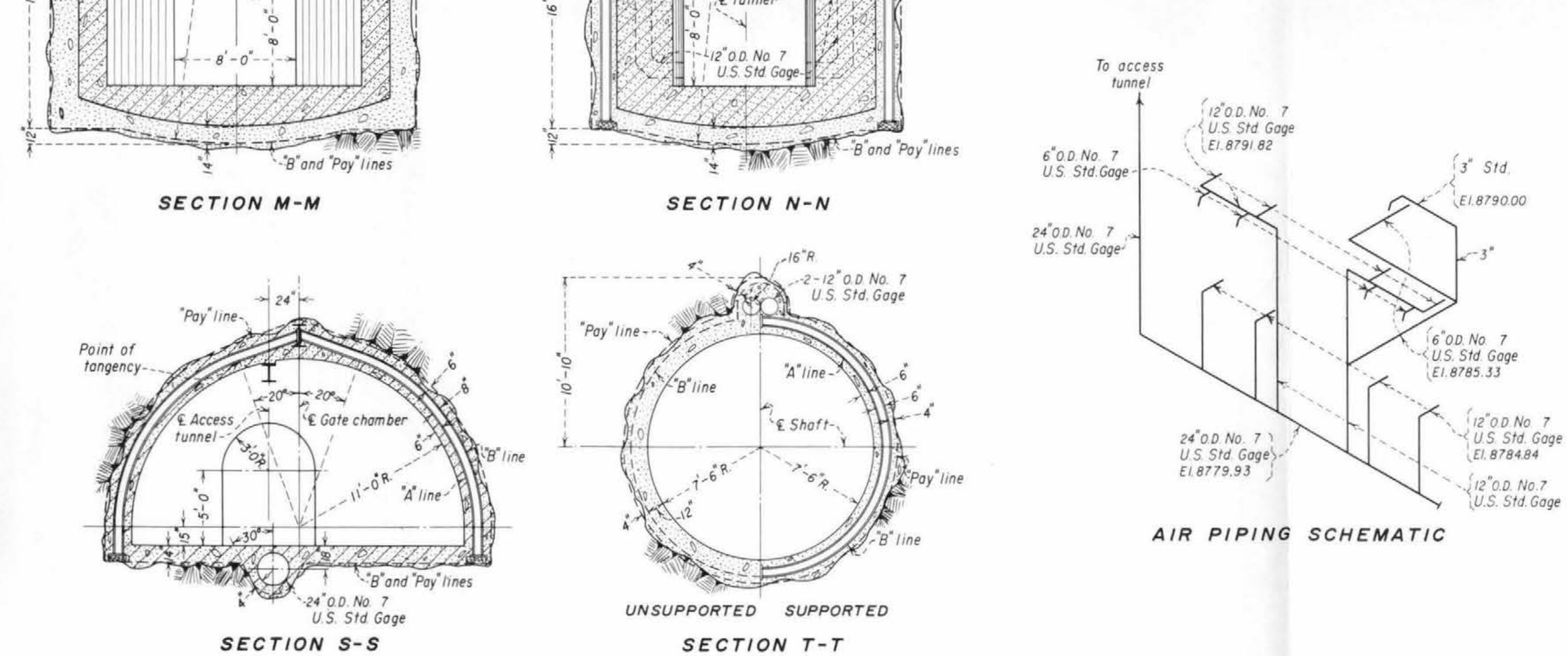
DILLON DAM
ALTERNATIVES 1, 2 AND 3
SHAFT SPILLWAY AND OUTLET WORKS
GENERAL PLAN AND PROFILE

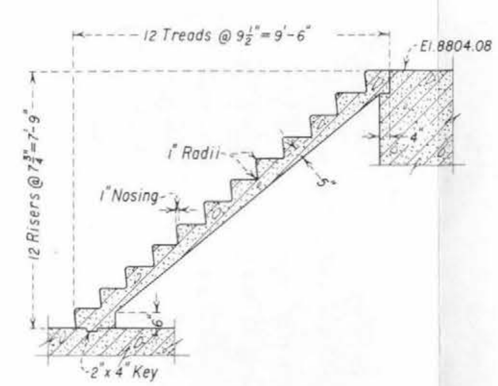
DRAWN G.Z.	APPROVED TIPTON AND KALMBACH, INC.
TRACED R.S.H.	
CHECKED L.I.V.	APPROVED BOARD OF WATER COMMISSIONERS

C-930 | TK35 | DENVER, COLORADO, JUNE, 1959 | SI-11

FIG. 120







NOTES

For locations of Sections, see Dwg. SI-20.
For Notes, see Dwg. SI-20.
Lagging behind steel supports not shown.

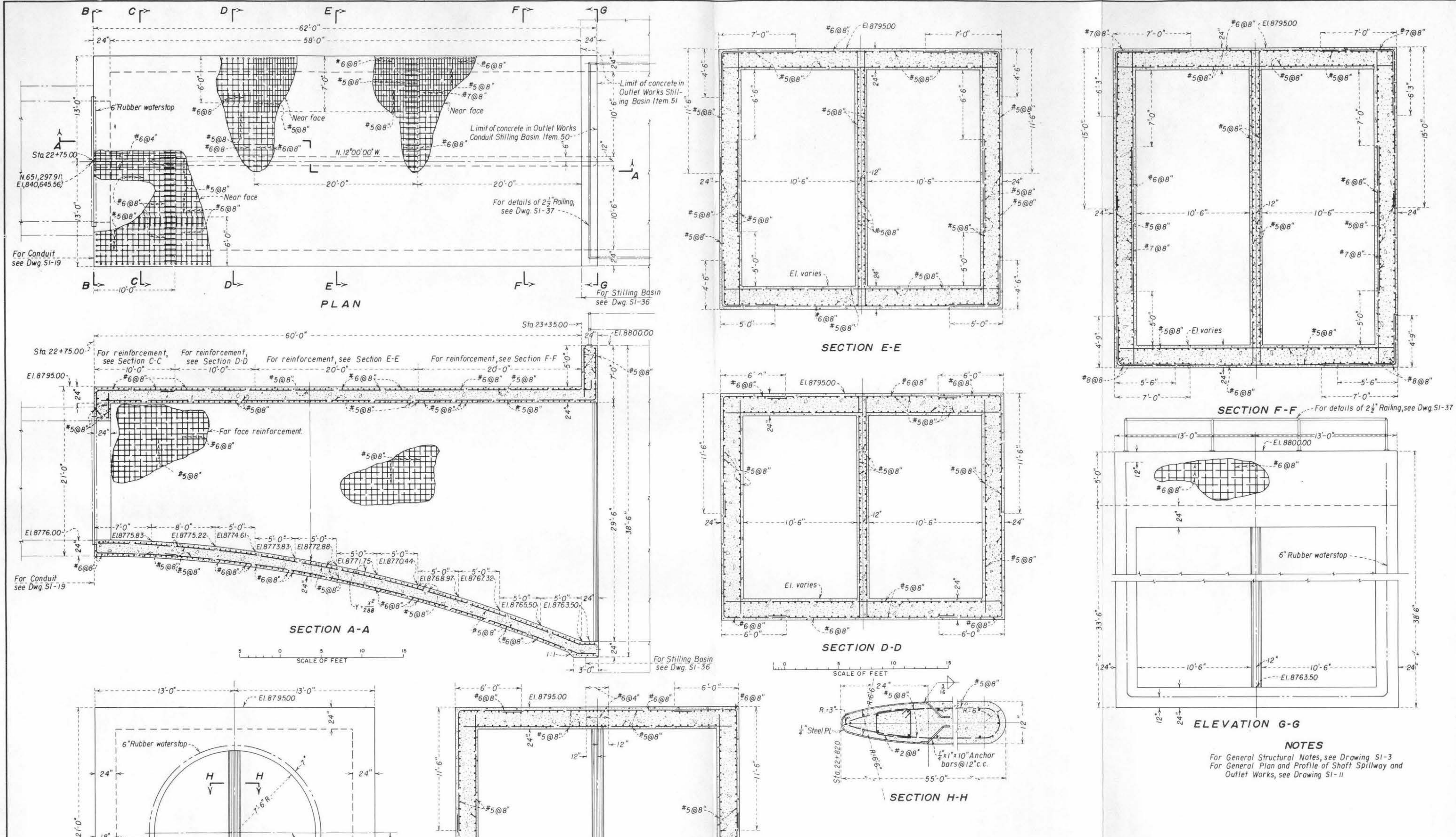
Approved on the 30TH day of JUNE 1959
J. E. Whitten
STATE ENGINEER

BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC., ENGINEERS
DILLON DAM
ALTERNATIVES 1, 2 AND 3
OUTLET WORKS GATE CHAMBER
SECTIONS

DRAWN J.C.E.	APPROVED	TIPTON AND KALMBACH, INC.	
TRACED R.S.H.			
CHECKED D.G.T.	APPROVED	BOARD OF WATER COMMISSIONERS	
TK35	DENVER, COLORADO,	JUNE, 1959	SI-21

C-930

FIG. 127



NOTES

For General Structural Notes, see Drawing S1-3
For General Plan and Profile of Shaft Spillway and Outlet Works, see Drawing S1-11

Approved on the 30TH day of JUNE 1959

J. E. Whitten
STATE ENGINEER

BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC., ENGINEERS
DILLON DAM
ALTERNATIVES 1, 2 AND 3
OUTLET WORKS STILLING BASIN TRANSITION
PLAN AND SECTIONS

DRAWN R.W.S.	APPROVED	TIPTON AND KALMBACH, INC.
TRACED G.Z.		
CHECKED K.D.H.	APPROVED	BOARD OF WATER COMMISSIONERS

C-930

TK35	DENVER, COLORADO, JUNE, 1959	S1-35

FIG. 128

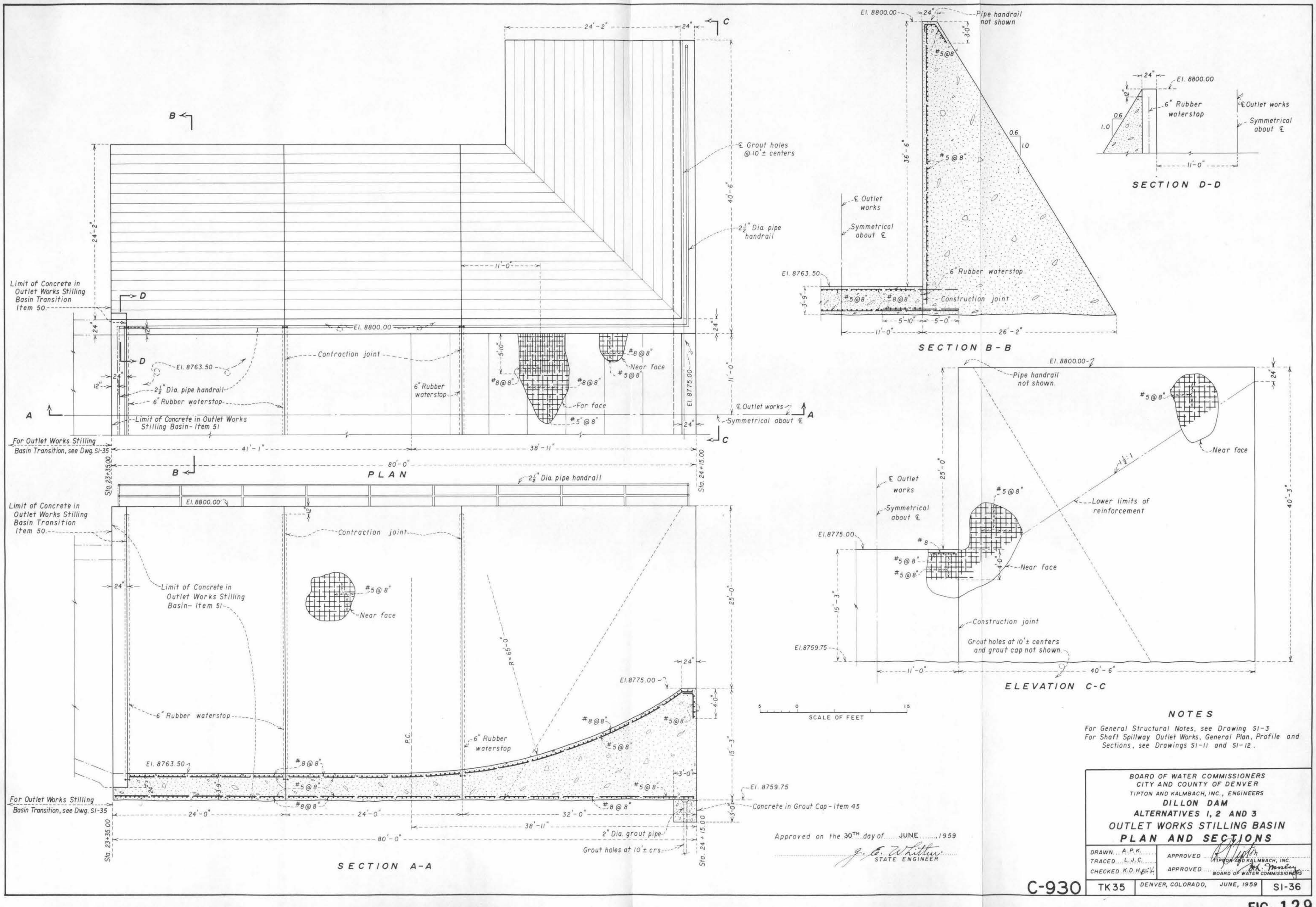

PLAN
SECTION A-A
SECTION B-B
SECTION D-D
ELEVATION C-C
Limit of Concrete in Outlet Works Stilling Basin Transition Item 50.
Limit of Concrete in Outlet Works Stilling Basin- Item 51
For Outlet Works Stilling Basin Transition, see Dwg. SI-35
℄ Grout holes @ 10' ± centers
$2\frac{1}{2}$" Dia. pipe handrail
6" Rubber waterstop
Contraction joint
El. 8800.00
El. 8763.50
El. 8775.00
El. 8759.75
℄ Outlet works
Symmetrical about ℄
Near face
Far face
#8 @ 8"
#5 @ 8"
Sta. 23+35.00
Sta. 24+15.00
41'-1"
38'-11"
80'-0"
24'-2"
40'-6"
11'-0"
24'-0"
32'-0"
25'-0"
15'-3"
R=65'-0"
P.C.
2" Dia. grout pipe
Grout holes at 10' ± crs.
Concrete in Grout Cap - Item 45
Pipe handrail not shown
Construction joint
36'-6"
26'-2"
5'-10"
5'-0"
0.6
1.0
11'-0"
Lower limits of reinforcement
Grout holes at 10' ± centers and grout cap not shown.
40'-3"
SCALE OF FEET
NOTES
For General Structural Notes, see Drawing SI-3
For Shaft Spillway Outlet Works, General Plan, Profile and Sections, see Drawings SI-11 and SI-12.
Approved on the 30TH day of JUNE 1959
J. E. Whitten
STATE ENGINEER
BOARD OF WATER COMMISSIONERS
CITY AND COUNTY OF DENVER
TIPTON AND KALMBACH, INC., ENGINEERS
DILLON DAM
ALTERNATIVES 1, 2 AND 3
OUTLET WORKS STILLING BASIN
PLAN AND SECTIONS
DRAWN A.P.K.
TRACED L.J.C.
CHECKED K.O.H.
APPROVED TIPTON AND KALMBACH, INC.
APPROVED BOARD OF WATER COMMISSIONERS
C-930
TK35
DENVER, COLORADO, JUNE, 1959
SI-36

FIG. 129